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RAPID-INFILTRATION SYSTEM FOR WASTEWATER RENOVATION AND BENEFICIAL REUSE

23rd Avenue Project, Phoenix, Arizona

by

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A full scale rapid infiltration wastewater treatment system consisting of four basins averaging 89 x 465 m. The hydraulic loading of 90 m/year was achieved by operating the basins in pairs with two weeks of wetting and two weeks of drying when secondary effluent with 10-15 mg/l suspended solids was applied. The hydraulic loading was drastically reduced when the suspended solids increased to 50-100 mg/l (mostly as algae) following detention in a stabilization pond.

The renovated water typically contained 750 mg/l dissolved solids, 0.9 mg/l suspended solids, 7 mg/l nitrogen (mostly nitrate), 0.25 mg/l phosphate-phosphorus, 20 fecal coliform per 100 ml, 1 virus PFU per 100 l and 2 mg/l total organic carbon. The organic compounds included trihalomethanes, chlorinated aliphatic and aromatic hydrocarbons, pesticides, and plasticizers.

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FOREWORD

The Environmental Protection Agency was established to coordinate the administration of major Federal programs designed to protect the quality of our environment.

An important part of the Agency's effort involves the search for information about environmental problems, management techniques, and new technologies which optimize the use of the nation's land and water resources while minimizing the threat of pollution.

EPA's Office of Research and Development conducts this search through a nationwide network of research facilities. As one of these facilities, the Robert S. Kerr Environmental Research Laboratory is responsible for the mangement of soil systems for the treatment and management of municipal wastewaters.

Although land application of municipal wastewater has been practiced for years, there has been a growing interest in recent years. The use of land application has received major impetus with the passage of the 1972 amendments to the Federal Water Pollution Control Act. The 1977 amendments gave further encouragement to the use of land application and provided incentives for the funding of these systems through the construction grants program. Rapid infiltration is one of the lesser developed land treatment technologies. Questions still exist regarding treatment effectiveness. This study demonstrates the ability of rapid infiltration to effectively treat wastewater beyond a conventional secondary step for unrestricted irrigation and primary contact recreation. This study also points out that some potentially carcinogenic organic compounds may move through rapid infiltration systems undegraded.

The report contributes to the knowledge which is essential for the EPA to meet the requirements of environmental laws and enforce pollution control standards which are reasonable, cost effective, and provide adequate protection for the American public.

Clinton W. Hall

Clinton W. Hall
Director
Robert S. Kerr Environmental Research Laboratory

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PREFACE

Renovating sewage effluent by groundwater recharge with rapid infiltration basins was started in the Salt River Valley on an experimental basis in 1967 with the Flushing Meadows Project. This project basically consisted of six parallel basins, 6.1 by 213 meter (m) each and 6.1 m apart, and observation wells to sample renovated water from the aquifer below the basins and at various distances away from the basins. The project came to a halt in the spring of 1978 when it was flooded and damaged by heavy runoff in the Salt River. In the more than 10 years of research with this project (Bouwer et al., 1980), the results have shown that:

1. The largest hydraulic capacity of the system was obtained with flooding periods of two to three weeks alternated with drying periods of 10 to 20 days. At this schedule and using a water depth of 0.3 m in the basins, the hydraulic loading rate was 122 m/year. Vegetation or gravel layers in the basins offered no particular advantages. Thus, the best bottom condition was bare soil. Volunteer vegetation seemed to have no undesirable effects, so weed control was unnecessary. Sludge-like accumulations on the basin bottoms were removed every one or two years. After 10 years of operating the project and a total infiltration of 754 m, there were no signs of reductions in hydraulic loading or in hydraulic conductivity of the underlying aquifer.

2. Nitrogen (N) removal from the effluent water as it seeped through the ground to become renovated water was about 30% at maximum hydraulic loading, but 65% if the loading rate was reduced to about 70 m/year by using 9-day flooding and 12-day drying cycles and by reducing the water depth in the basins to 0.15 m. The form and concentration of N in the renovated water sampled from the aquifer below the basins were slow to respond to a reduction in hydraulic loading (Bouwer, et al. 1980). In the 10th year of operation (1977), the renovated water contained 2.8 mg/ℓ ammonium-N (NH_4-N), 6.25 mg/ℓ nitrate-N (NO_3-N), and 0.58 mg/ℓ organic-N, for a total N-content of 9.6 mg/ℓ. This was 65 percent less than the total N-content of the secondary sewage effluent, which averaged 27.4 mg/ℓ (mostly as NH_4-N) in that year.

3. Phosphate (PO_4) removal increased with increasing distance of underground movement of the sewage water. After 9 m of downward movement to and in the underlying aquifer, removal was about 40% at high hydraulic loading and 80% at reduced hydraulic loading. Additional lateral movement of 61 m through the aquifer increased the removal to 94% in 1977 (i.e., a concentration of 0.51 mg/ℓ PO_4-P in the renovated

water versus 7.9 mg/ℓ in the effluent). After 10 years and a total infiltration of 754 m, there were no signs of a decrease in PO_4 removal.

4. Fluoride removal paralleled PO_4 removal. Fluoride concentrations in 1977 were 2.08 mg/ℓ for the effluent, 1.66 mg/ℓ for renovated water sampled from wells between the basins, and 0.95 mg/ℓ for renovated water sampled from a well 30 m away from the basins.

5. Boron was not removed and was present at concentrations of 0.5 to 0.7 mg/ℓ (0.59 mg/ℓ in 1977) in the effluent and renovated water.

6. Concentrations of zinc, copper, cadmium, and mercury in effluent and renovated water were below maximum limits for drinking and permanent irrigation. The concentration of lead in the renovated water was 0.066 mg/ℓ, which exceeded the 0.05 mg/ℓ limit for drinking water but was well below the 5 mg/ℓ limit for permanent irrigation.

7. The biochemical oxygen demand (BOD) was reduced from a range of 10 to 20 mg/ℓ for the effluent to essentially zero for the renovated water. Corresponding ranges for chemical oxygen demand (COD) were 30 to 60 mg/ℓ in the effluent and 10 to 20 mg/ℓ in the renovated water. Not all organic carbon was removed from the effluent water as it seeped through the soil and aquifer. The total organic carbon (TOC) contents of the renovated water averaged 5.2 mg/ℓ for the wells between the basins and 3.3 mg/ℓ for the wells 30 m away from the basins. The average TOC content of the secondary sewage effluent was 19.2 mg/ℓ. Trace organics in renovated wastewater presently are the main concern in potable use of such water.

8. The secondary sewage effluent contained 21 virus plaque forming units (PFU's) per liter, but viruses could not be detected in renovated water sampled below the basins (Gilbert et al., 1976). Fecal coliforms were present at concentrations of up to several hundred per 100 mℓ in the renovated water sampled below the basins, but could not be detected in renovated water sampled 91 m away from the basins. Hence, a lateral movement of about 100 m through the aquifer apparently produced renovated water free from fecal coliforms. Laboratory studies showed that distilled water (rain!) could remobilize previously adsorbed viruses in the soil only during the first 24 hours of a drying period, but that addition of calcium chloride to the water prevented most of this desorption. This finding is important for managing rapid-infiltration systems in humid climates.

Preliminary cost estimates indicated that most of the cost of renovating sewage effluent by rapid-infiltration for groundwater recharge was in pumping it from the wells. This presently costs about \$10 to \$20/1000 m^3 or acre-foot for the relatively high groundwater levels (5- to 20-m depth) in and near the Salt River bed.

The 23rd Avenue Project gave an opportunity to extend the results obtained at the Flushing Meadows Project to a larger facility and to evaluate how hydraulic loading and quality improvement of the effluent water as it moved through the underground materials were affected by the much larger infiltration basins, coarser soils, more algae in the sewage effluent, deeper groundwater levels, and deeper movement in the aquifer. At the same time, the project would enable the development of design and management criteria for large systems, and it would produce enough renovated water for commercial reuse by an irrigation district.

ABSTRACT

An oxidation pond of 16 hectare (ha) below the Phoenix 23rd Avenue Sewage Treatment Plant was split into four parallel infiltration basins to study large-scale renovation of secondary effluent (activated sludge, unchlorinated). Surface soils were sands and gravels and groundwater levels were at depths of 3 to 20 meters (m). Initially, the infiltration basins received effluent after it had passed through a 32-ha lagoon. Algal blooms in this lagoon, however, caused low infiltration rates in the infiltration basins (hydraulic loading 22 m/year). This problem was alleviated with a bypass channel around the lagoon, which increased the hydraulic loading rate of the infiltration basins to 90 m/year. This was 14% of the hydraulic conductivity of the surface soils. Air pressures in the vadose zone increased when the basins were flooded, but they had no significant effect on infiltration rate. Shallow basins were preferable to deep basins because higher turnover rates of the effluent water reduced growth of suspended algae and associated soil clogging. Transmissivity and specific yield of the aquifer were evaluated with different techniques. Groundwater levels rose about 1 m in response to recharge. A production well pumping renovated water from 30- to 55-m depth in the center of the project yielded better quality renovated water than shallow wells. The renovated water typically contained 750 mg/ℓ dissolved solids, 0.9 mg/ℓ suspended solids, 7 mg/ℓ nitrogen (almost all as nitrate), 0.25 mg/ℓ phosphate-phosphorus (PO_4-P), 1 to 20 fecal coliforms per 100 mℓ (average yearly values; much lower values can be expected as the plant will start chlorinating its effluent in late 1980), about 1 PFU of viruses per 100 ℓ, and 1.8 mg/ℓ total organic carbon. Except for the dissolved solids, these values were much lower than those for the secondary effluent entering the ground. Total organic carbon included a wide variety of organic compounds, including trihalomethanes (total concentration less than 0.1 mg/ℓ), and other organohalides, pesticides, plasticizers, and wood preservatives. Concentrations generally were on the order of nanograms to micrograms/ℓ. Significant losses of volatile organic compounds occurred in the infiltration basins. Some compounds were also significantly attenuated in the soil-aquifer system. The renovated water will be suitable for unrestricted irrigation and for lakes with primary contact recreation.

This report was submitted in fulfillment of Project No. S-802435 by the City of Phoenix under sponsorship of the U.S. Environmental Protection Agency. The research was done by the U.S. Water Conservation Laboratory of the U.S. Department of Agriculture, Phoenix, Arizona. The report covers the period July 1975-October 1980, and was completed as of December 1980.

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LIST OF ABBREVIATIONS AND SYMBOLS

mm -- millimeter
cm -- centimeter
m -- meter
km -- kilometer
ha -- hectare
ℓ -- litre
M -- mole
kg -- kilogram
g -- gram
mg -- milligram
µg -- microgram
ng -- nanogram
TDS -- Total dissolved solids
TOC -- Total organic carbon
PFU - Plaque forming unit

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SECTION 1

INTRODUCTION

An inseparable byproduct of the "population explosion" in the sunny southwest is a "sewage explosion". Many areas have inadequate water resources and groundwater depletion is rampant. The greater Phoenix area produced about 130 million cubic meters (m^3) of sewage effluent per year toward the end of the 1970's. The annual rate of increase in the sewage flow is about 6 million m^3/year. In the near future most of this flow or about 70 million m^3/year will be used for cooling by a nuclear power plant. The rest, however, could be used for irrigation and also for recreational lakes and wildlife sanctuaries. The experimental Flushing Meadows Project in the Salt River bed west of Phoenix had shown that the secondary effluent (activated sludge) from the 91st Avenue Sewage Treatment Plant could be effectively and inexpensively upgraded by a rapid-infiltration system with recovery of the renovated water from the underlying aquifer by wells. The resulting "renovated" water met the requirements for unrestricted irrigation or for lakes with primary-contact recreation. The results also incidated that about 200 hectare (ha) of infiltration basins would be necessary to infiltrate the sewage effluent from about one million people, and that the infiltration basins could best be located along both sides of the normally dry Salt River bed. Wells with submersible pumps would then be drilled in the center of the river bed to pump the renovated water out. Renovated water would be pumped at such a rate that encroachment of the renovated water into the aquifers adjacent to the river bed would be minimized. The present study was conducted to see (1) how large-basin rapid-infiltration systems should be managed for optimum hydraulic loading, (2) what quality improvement of the effluent water could be obtained if it infiltrated in coarser textured soils and moved to deeper aquifers and wells (including viruses and trace organics), and (3) how the vadose zone and the groundwater levels would respond to infiltration from large basins.

SECTION 2

CONCLUSIONS

1. Algal blooms in secondary sewage effluent severely reduced hydraulic loading rates in rapid-infiltration basins. Hydraulic loading rates were only 22 m/year when the secondary effluent first went through an oxidation pond with a detention time of three days, but increased to a projected 90 m/year (14% of hydraulic conductivity of surface soil) when the oxidation pond was bypassed and effluent flowed directly from the treatment plant into the infiltration basins. The main algal bloom organisms was Carteria klebsii Dillworth, which is a single-cell type algae that occured in the water mostly in suspended form. These algae not only formed a "filter cake" on the bottom of the infiltration basins, they could also cause precipitation of calcium carbonate on the bottom due to uptake of carbon dioxide for photosynthesis and resulting increase in pH. Suspended solids contents of the algae-laden effluent water from the oxidation pond often were in the range of 50 to 100 mg/ℓ in the summer. After construction of the bypass channel, suspended solids contents of the effluent entering the infiltration basins generally were in the range of 10 to 15 mg/ℓ. Algal growth in the infiltration basins themselves was minimized by using small water depths [10 to 20 centimeters (cm)] to promote fast turnover of the water in the basins. Large water depths (1 m or more) increased the hydraulic gradient for infiltration, but in the summer this increase was more than offset by soil clogging due to additional algae growth. When small water depths were used, algae still grew on the bottom (mostly Oscillatoria) or floated on the surface (mostly Chlamydomonas and Pandorina). The effluent water itself, however, remained quite clear and the bottom algae did not seem to have a significant effect on infiltration rates.

2. Bottoms of infiltration basins must be carefully graded to avoid low spots where water could be standing for long times during drying. Such spots would either require very long drying periods, or, if normal drying periods are used, infiltration rates are not restored for the areas of the low spots. Either way, hydraulic loading rates of the basins will be reduced.

3. Ripping the bottoms of the infiltration basins with a grader just about doubled hydraulic loading rates. The cost of the additional hydraulic capacity of the system obtained in this manner was about $0.10 per 1000 m^3 or per acre-foot of additional infiltration.

4. The four infiltration basins usually were operated in pairs (two basins flooded and two basins drying), rotating flooding and drying every one to two weeks. When infiltration rates were very high, only one basin could be flooded at a time and the cycle was one week flooding-three

weeks drying. This cycle was also used during certain times in the winter when drying periods needed to be longer than flooding periods to obtain adequate recovery in infiltration rates.

5. The design hydraulic loading rate of the rapid-infiltration system can be taken as 90 m/year. This is 14% of the hydraulic conductivity of the surface soils. Thus, the 16-ha rapid infiltration system below the 23rd Avenue Tratment Plant can renovate 14.4 million m^3 (12,000 acre-feet) of effluent water per year, or almost 40,000 m^3 per day (32 acre-feet per day, 11 million gallons per day, or 7600 gallons per minute).

6. Air pressures in the lower vadose zone (at 12.2-m depth) rose in the beginning of a flooding period, but decreased to essentially zero toward the end of the flooding period. Initial air-pressure increases varied linearly with initial infiltration rates, and were about 7.2 m of water per m/day infiltration rate. Thus, an infiltration rate of 0.4 m/day at the beginning of an inundation period produced a temporary increase in the air pressure at 12.2-m depth in the vadose zone of 2.9 m. The air pressure buildups in the vadose zone did not seem to significantly affect infiltration rates.

7. Depth to the groundwater table varied from 3 to 20 m, depending on flow and recharge events in the normally dry Salt River south of the basins, and on groundwater pumping north of the project. The natural groundwater slope was from south to north (probably more to the northwest) at gradients of 0.2 to 0.8%. Groundwater levels responded to infiltration from the basins. Typically, the groundwater levels in the center would rise about 1 m in response to flooding the two center basins for 10 days during which the average infiltration rate was 0.33 m/day. Drying periods produced declines in groundwater levels.

8. Hydraulic properties of the aquifer beneath the basins were determined with the slug test, drawdown test, and Theis pumping test on the Center Well. They were also calculated from the rise of the groundwater in the center of the basin area in response to infiltration from the two adjacent basins. The methods yielded different values for transmissivity and specific yield, as can be expected due to differences in methodology and in the portions of the aquifer and the flow directions therein "covered" by the tests. The results indicated that a transmissivity of 50,000 m^2/day and a specific yield of 0.05 seemed reasonable for design purposes.

9. Suspended solids concentrations of the renovated water from the production well in the center averaged 0.9 mg/ℓ in 1979 and 1980. The dissolved solids content averaged 738 mg/ℓ, which was about 40 mg/ℓ more than that for the secondary effluent. This increase was due to evaporation from the basins and possibly also to dissolution of calcium carbonate in the soil. The effluent from the treatment plant was slightly alkaline (pH of 8, increasing to 9 and 10 during algal blooms), while the renovated water was essentially neutral (pH of 7).

10. The average total-nitrogen (N) concentration of the renovated water from the production well in the center of the project was 7.2 mg/ℓ in 1980, while that of the shallower wells in the project area was 9 mg/ℓ. Most of this N was in the nitrate form. The average total-N content of the secondary effluent in 1980 was 18 mg/ℓ. Thus, the soil-aquifer system removed about 60% of the N in the effluent water. In other years, N removals were 61% (1979) and 75% (latter part of 1976). Detailed studies on N transformations in the soil using the acetylene inhibition method showed that most of the denitrification in the top 5 cm of the soil occurred in the first two or three days of a drying period, and that this accounted for an N-removal of about 10% in the summer and 20% in the winter.

11. Phosphate-phosphorus (PO_4-P) concentrations averaged about 6 mg/ℓ in the secondary effluent, 1 to 3 mg/ℓ in the renovated water from the shallow wells, and 0.25 mg/ℓ in the renovated water from the production well in the center of the project. PO_4-P removal increased with increasing distance of movement of the water through the underground materials (horizontally as well as vertically) and with decreasing infiltration rates.

12. Fluoride concentrations averaged 1.2 mg/ℓ in the secondary effluent and 0.7 mg/ℓ in the renovated water from the production well in the center. Boron concentrations were 0.5 mg/ℓ for effluent and renovated water. Since the Flushing Meadows Project had already indicated that heavy metal concentrations in the effluent and renovated water were below maximum limits for permanent irrigation, additional metal analyses were not made.

13. The fecal coliform concentration of the secondary effluent averaged between 1×10^6 and 2×10^6 per 100 mℓ. This concentration was reduced to about 10^4 per 100 mℓ after the effluent flowed through the 32-ha oxidation pond. Thus, fecal coliform concentrations of the effluent entering the infiltration basins were about 10^4 per 100 mℓ before the bypass channel around the 32-ha pond was constructed, and 1 to 2×10^6 thereafter. Initially, fecal coliform concentrations in the renovated water were quite low. They exhibited peaks of 10 to 40 per 100 mℓ when water that had infiltrated at the beginning of a flooding period reached the wells, but averaged 1 to 2 per 100 mℓ for the rest of the time. The higher infiltration rates in 1980, however, also produced higher coliform counts in the renovated water, particularly for the wells that tapped the top of the groundwater (one such well showed peaks of more than 1000 per 100 mℓ and one peak of even 17,000 per 100 mℓ). The average for the production well in the center was 22 per 100 mℓ. This concentration can be reduced by scheduling flooding and drying cycles so that the basins on each side of the production well are never flooded at the same time. Also, production wells could be perforated deeper or, if other systems are built, they could be drilled a distance away from the infiltration basins. For the 23rd Avenue Project, however, much lower fecal coliforms in the effluent and renovated water can be expected after 1 January 1981, when the chlorination facility of the treatment plant will be on line.

14. Virus concentrations in the renovated water from shallow wells in the project averaged 0.1 plaque forming units (PFU)/100 ℓ in January 1980 and 1.75 PFU/100 ℓ in June 1980 when a different technique was used to concentrate the viruses from a sample of 800-2000 ℓ into a small volume suitable for assay. At that time, the production well yielded water with a virus content of 1.3 PFU/100 ℓ. Virus strains identified were Coxsackie types B3 and B4, and Polio type 1.

15. Concentrations of total organic carbon (TOC) averaged 10.5 mg/ℓ in the secondary effluent, 2.3 and 3.3 mg/ℓ in the renovated water from two shallow wells, and 1.8 mg/ℓ in the renovated water from the deeper production well in the center. Thus, additional movement through the aquifer reduced the TOC content of the water. The TOC values of the renovated water represent organic compounds that are not readily biodegradable. Identification of these trace organics showed a wide variety of compounds, including trihalomethanes, numerous other organohalides, aromatic compounds, anisoles, phthalates, and pesticides. Concentrations often were on the order of nanograms and micrograms/ℓ. The total concentration of trihalomethanes in the renovated water was below the interim maximum limit of 0.1 mg/ℓ for drinking water. Significant losses of volatile organics took place in the basins themselves. Some compounds were significantly attenuated in the soil-aquifer system whereas others were not. More systematic studies are planned for the future, including the effect that chlorination of the effluent in the treatment plant will have on the trace organics and their fate in the soil-aquifer system.

16. The 23rd Avenue Project has demonstrated that large-scale renovation of conventionally treated sewage effluent by rapid-infiltration systems and recovery of renovated water from wells is feasible. Since most of the cost of renovating the effluent with this system is in pumping the water from the wells, it is also cost effective. The renovated water is suitable for unrestricted irrigation and water-based recreation. Further treatment could also make it suitable for drinking.

SECTION 3

RECOMMENDATONS

Two more production wells should be installed on the center dike so that the pumping capacity of the project equals the infiltration capacity. The wells should be deeper (total depth about 150 m, for example, if no clay layers or other restricting zones are encountered) and the pumps should be set deeper than for the existing production well to allow greater drawdown of the water level and to maintain pumping capacity during extended periods of drought when groundwater levels in the Valley as a whole decline. When the three wells are in, studies should be made of how infiltration and pumping rates should be managed to minimize movement of renovated water into the aquifer adjacent to the infiltration system. Plans should also be made for the beneficial use of the renovated water, such as unrestricted irrigation or lakes with primary-contact recreation.

If the demand for renovated water increases and sufficient effluent is available, additional infiltration basins can be constructed in the 32-ha area east of the four existing infiltration basins that was originally also used as an oxidation pond. Dikes should be constructed north-south, effluent can be admitted from the bypass channel on the south side of the area, and production wells can be drilled in the centers of the dikes, thus extending the existing line of three wells eastward. Studies should be made to determine if chlorination of the secondary effluent used for rapid infiltration should be discontinued, or could be more effectively applied to the discharge point of the renovated water.

Further studies should be made of the trace organics in the effluent and renovated water, including the effect of chlorination on formation of additional organohalides, attenuation in the soil-aquifer system, and possible stimulation of this attenuation by increasing the organic carbon content of the effluent to promote secondary utilization. Uptake of trace organics by plants and possible concentration effects of trace organics in crops should also be studied to determine the suitability of sewage effluent and renovated sewage water for irrigation of edible crops.

More research needs to be done to establish the optimum combination of the sewage treatment before rapid infiltration, the quality improvement obtained in the soil-aquifer system, and the possible treatment of the renovated water. Primary treatment with additional clarification to reduce the suspended solids content (including the possible use of chemicals such as lime or coagulants) may well be adequate, since the soil-aquifer system can handle more dissolved biochemical oxygen demand than that of secondary effluent. A higher organic-carbon content of the infiltrating effluent actually may be preferable, because it could increase denitrification and breakdown of refractory organics by secondary utilization.

Institutional and social constraints on the effective, beneficial, and most economical reuse of wastewater should be reduced.

SECTION 4

DESCRIPTION OF SYSTEM

The 23rd Avenue Rapid Infiltration Project was constructed in the lower pond of a two-lagoon, in series, system that had been used to stabilize the effluent of the 23rd Avenue Sewage Treatment Plant (Figure 1). The lagoons were 32 ha and 16 ha in size and they were constructed on the north side of the Salt River bed, east of 35th Avenue. They became necessary in the 1950's when population growth in the Phoenix area created problems of overloading at the 23rd Avenue Sewage Treatment Plant. After construction of the 91st Avenue Sewage Treatment Plant in the early 1960's and expansion of the 23rd Avenue plant, the latter again was operating at design capacity and the oxidation ponds were no longer necessary. Since interest had been developing in an operational rapid infiltration system with recovery of the renovated water for unrestricted irrigation, the old lagoons offered an excellent opportunity for the construction of such a system. Three dikes were built in the lower, 16-ha oxidation pond to split it into four infiltration basins of approximately 4 ha each. The average size of the basins was 89 x 465 m. The bottom areas were 4.25 ha for basin 1, 4.13 ha for basin 2, 4.09 ha for basin 3, and 4.13 ha for basin 4 (Figure 1).

The surface soil in most of the Salt River bed typically consisted of fine, loamy sand. Gravel and coarse sand layers prevailed below a depth of about 1 m (Figure 2). Since the soil material for constructing the dikes of the infiltration basins in the lower oxidation pond was taken from within the pond area, some of the underlying gravel and coarse sand strata were exposed. Thus, the surface soil of the basins consisted of loamy sand, coarse sand, and coarse sand plus gravel and boulders (Figure 3). Below 0.5 m, gravel and coarse-sand strata prevailed to a depth of at least 60 m (the depth of the deepest well drilled in the project area). The depth of the groundwater table generally varied from 3 to 20 m, depending on periodic natural recharge from the Salt River and seasonal groundwater pumping in the area. The water table is the upper boundary of an unconfined aquifer that extends to great depth. As in any stratified alluvial material, some perching zones may exist in the vadose zone. The cuttings from the well drilling operations, however, did not indicate any major perching layers.

Unchlorinated secondary effluent (activated sludge process) from the 23rd Avenue plant entered the 32-ha lagoon on the east side, then flowed through the lagoon (Figures 3 and 4), and entered the infiltration basins via culverts and rectangular weirs (Figure 5) to monitor

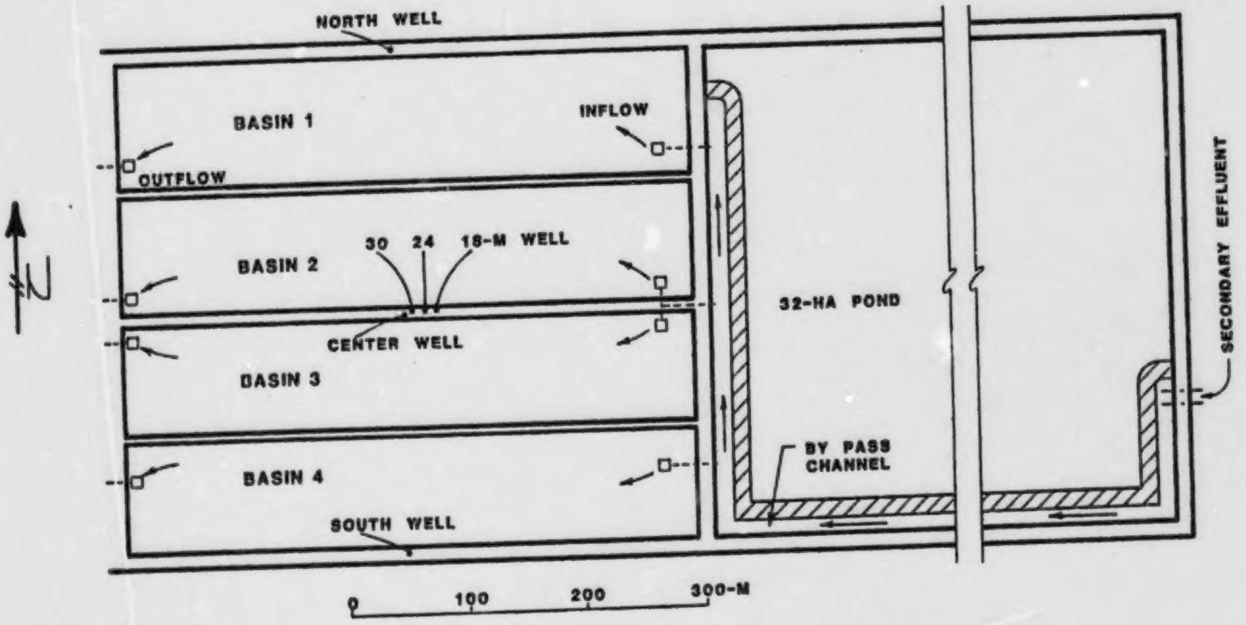
NORTH WELL
BASIN 1
INFLOW
OUTFLOW
BASIN 2
30
24
18-M WELL
CENTER WELL
BASIN 3
BASIN 4
SOUTH WELL
32-HA POND
BY PASS CHANNEL
SECONDARY EFFLUENT
N
0
100
200
300-M

Figure 1. Schematic of 23rd Avenue Rapid Infiltration System.

Figure 2. North wall of gravel pit north of basin 1 indicating sand and gravel layers typical of the "river run" material in the area.

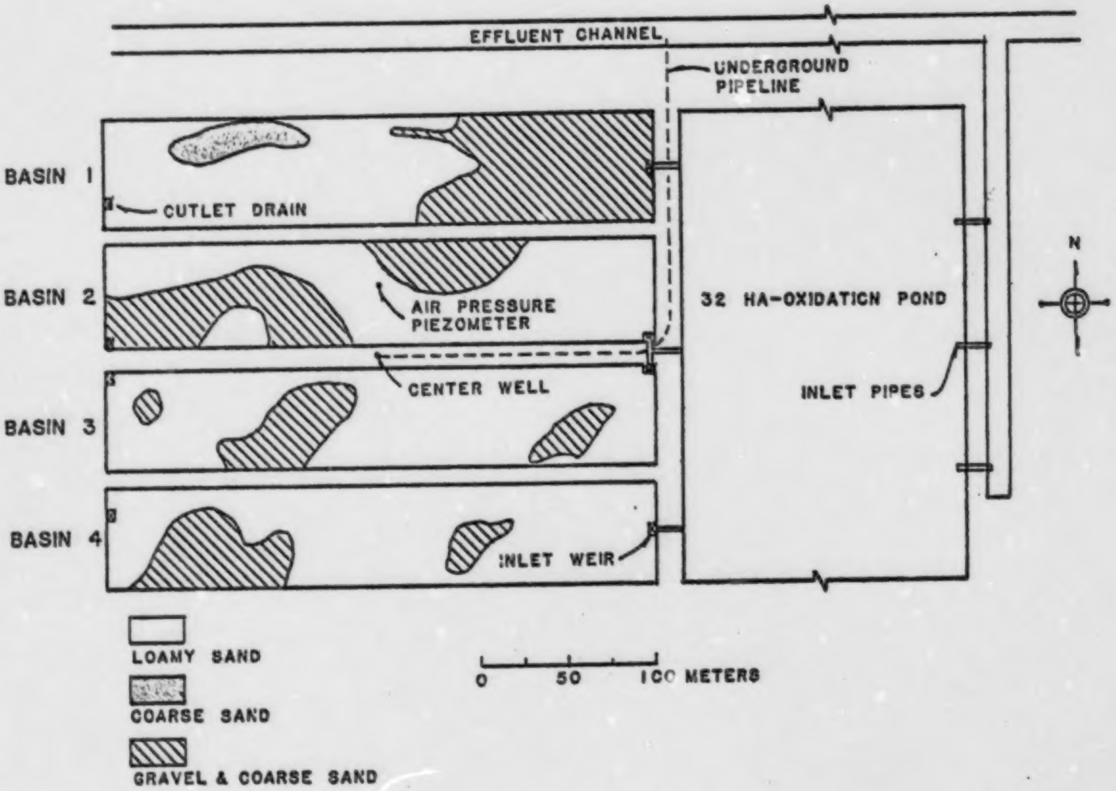
EFFLUENT CHANNEL
UNDERGROUND PIPELINE
BASIN 1
OUTLET DRAIN
BASIN 2
AIR PRESSURE PIEZOMETER
32 HA-OXIDATION POND
CENTER WELL
INLET PIPES
BASIN 3
BASIN 4
INLET WEIR
N
LOAMY SAND
COARSE SAND
GRAVEL & COARSE SAND
0
50
100 METERS

Figure 3. Major types of surface soil in the infiltration basins. The dashed line represents the underground pipeline that takes the renovated water from the Center Well to the effluent channel north of Basin 1.

Figure 4. Aerial view of the four infiltration basins (left) and the 32-ha lagoon (right) with Phoenix in background.

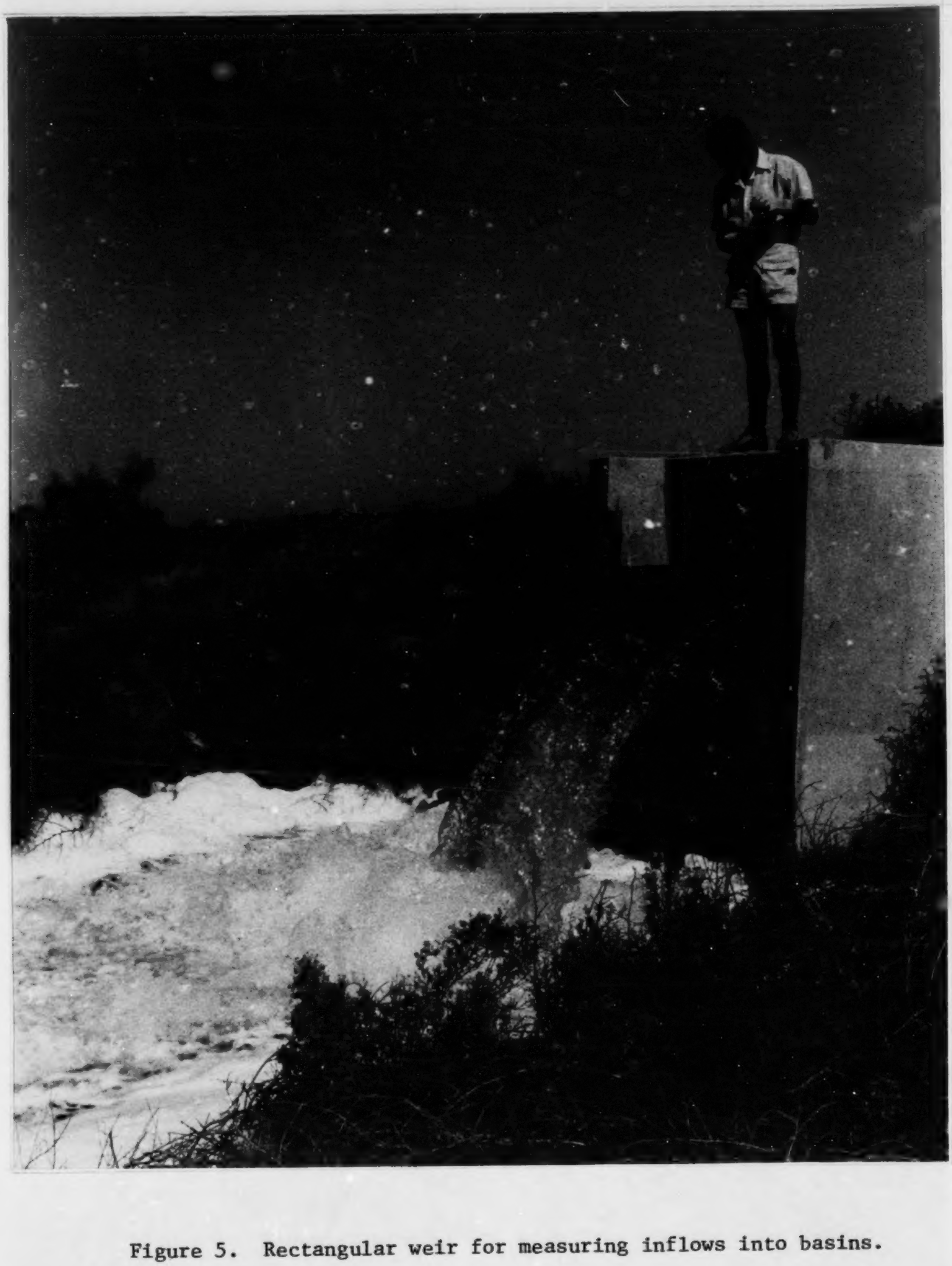

Figure 5. Rectangular weir for measuring inflows into basins.

inflow rates. The water depth in the basins was controlled by a rectangular outflow weir on the west side of each basin (left in Figure 1), which also enabled monitoring of the outflow rates. The outflow weirs were mounted on vertical slide gates, which were opened when a drying period was started to rapidly drain the effluent water from the basins.

The flow of the effluent through the 32-ha lagoon created problems of algae growth in the summer. The detention time in this lagoon was about three days, which was long enough to produce intense algal blooms. Thus, while the inflow into the 32-ha lagoon had a low suspended solids content, especially in the summer, the water flowing out of the 32-ha lagoon and into the infiltration basin had a much higher suspended solids content due to algae, and looked very much like pea soup. The algae caused severe clogging of the bottoms of the infiltration systems, resulting in low infiltration rates and hydraulic loading. To solve this problem, a bypass channel was constructed in 1979, on the south side of the 32-ha lagoon to transport secondary effluent directly into the infiltration basins (Figures 1 and 6). This greatly reduced the opportunity for algae growth and markedly increased the infiltration rates in the basins. After completion of the bypass channel, the 32-ha lagoon was kept dry, except for some leakage around the old inlet culverts on the east side (Figure 6).

In the same year that the infiltration basins were constructed (1975), two observation wells and one production well were installed using the cable-tool method. The observation wells were 23 m in depth and they were cased with 15-cm diameter steel pipe to the bottom. The wells were located half-way down the infiltration basins, one on the north side (North Well) and one on the south side (South Well; Figure 1). The production well (Center Well) was drilled in the center of the the basin area (Figure 1). The well is 61 m in depth and it is cased with 61-cm diameter steel pipe to the bottom. The casing was perforated in place with a louver-type perforator from 30- to 55-m depth. The well was equipped with a turbine pump and electric motor (Figure 7) to pump about 13,000 m^3/day. The pumped water was conveyed by an underground pipeline to the northeast corner of the basin area, where it was discharged into the effluent channel on the north side of the project (Figure 3). The pipeline needs to be extended about 1.2 km north to an irrigation ditch if and when the renovated water will be used for unrestricted irrigation by the Roosevelt Irrigation District. Two more wells must then be installed on the center dike to pump renovated water. This will bring the production of renovated water up to the infiltration rate.

Three additional observation wells were drilled near the Center Well in December 1979. The purpose of these wells was to sample renovated water from the top of the aquifer for analysis of trace organics, microorganisms, and other constituents. Because of the fluctuating water table, wells at different depths were necessary to allow sampling as close to the water table as possible to get a representative sample of renovated water as it arrived at the groundwater. To avoid local contamination of the aquifer with drilling mud or other

Figure 6. Aerial view of bypass channel (left) to convey secondary effluent directly to the four infiltration basins.

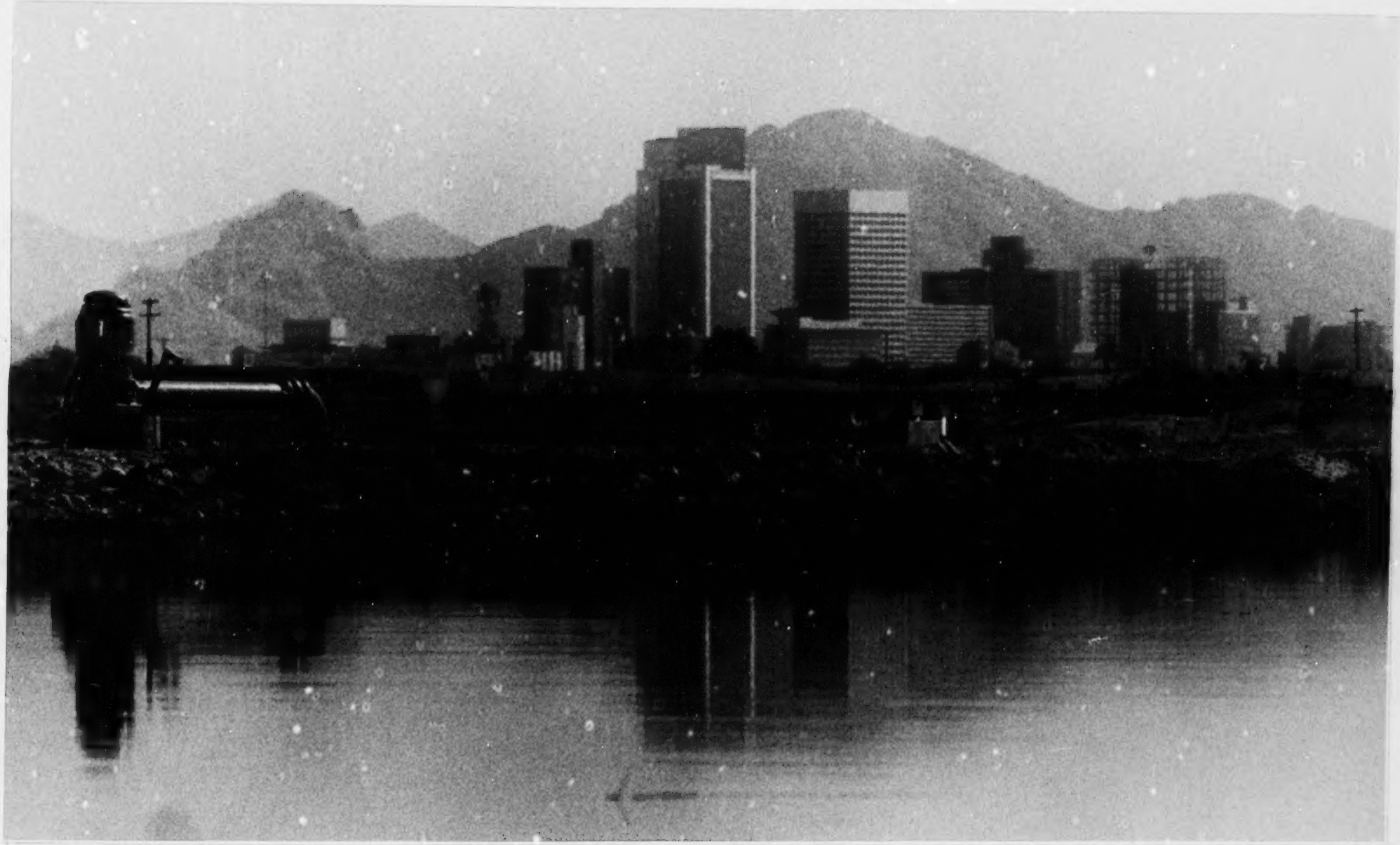

Figure 7. Center Well with basin 3 in foreground and downtown Phoenix in background.

additives, the wells were drilled with the air-rotary method. The wells were 18.3, 24.4, and 30.5 m in depth, respectively, and they were cased with 15-cm diameter steel pipe to the bottom. The wells were located on a line 6 m north of the Center Well at radial distances from the Center Well of 22.4 m for the 30-m Well, 25.3 m for the 24-m Well, and 28.6 m for the 18-m Well (Figure 1). All observation wells were equipped with domestic submersible pumps that had a capacity of about 1 ℓ/second and were operated with a portable generator.

SECTION 5

INFILTRATION STUDIES

BASIN INFILTRATION RATES WITH SECONDARY EFFLUENT FROM LAGOON

Since the algae in the sewage effluent from the 32-ha lagoon were expected to produce soil clogging in the infiltration basins, a relatively large water depth of 0.9 m was initially selected for the basins with the thought that water depth would compensate for clogging effects and that reasonable infiltration rates could still be obtained. The results, however, proved this wishful thinking to be wrong. Infiltration rates were quite low and further experimentation showed that the only way to get higher infiltration rates was to use sewage effluent with a lower algae content. For this purpose a bypass channel was constructed later in the 32-ha lagoon so that secondary effluent could flow directly into the infiltration basins without having to go first through the lagoon.

In the initial studies, with the effluent going through the 32-ha lagoon before it entered the infiltration basins, the basins were without vegetation. To maintain symmetry in the underground flow system, the basins were flooded and dried in pairs symmetric with respect to the center dike (i.e. basins 1 and 4 flooded and basins 2 and 3 drying, and vice versa). The length of the flooding and drying periods was mainly determined by how fast the infiltration rates declined, how long it took for the water to drain out of the basins, and how fast the bottom soil dried. The following schedules were used:

Period	Flooding (weeks)	Drying (weeks)
Jul-Dec 1975	2	2 to 4
Jan-May 1976	2	2
Jun-Sep 1976	1	4

The infiltration rates (calculated from inflow, outflow, and water level changes) in 1975 generally were between 0.3 and 0.6 m/day initially, but dropped to less than 0.15 m/day after two weeks of flooding (Figures 8, 9, 10, and 11). Infiltration rates remained fairly high (about 0.6 m/day) in basins 2 and 3 for the first week of flooding in January 1976 when the suspended solids content of the effluent from the 32-ha lagoon was relatively low (Figure 12). For the rest of 1976, infiltration rates showed rapid declines to values often less than 0.08 m/day. Infiltration rates averaged about 0.15 m/day for the flooding periods. By midsummer, the algae concentration in the effluent from the 32-ha lagoon was so high (> 50 mg/ℓ) and caused so much clogging of the soil surface in the infiltration basins, that it became necessary to

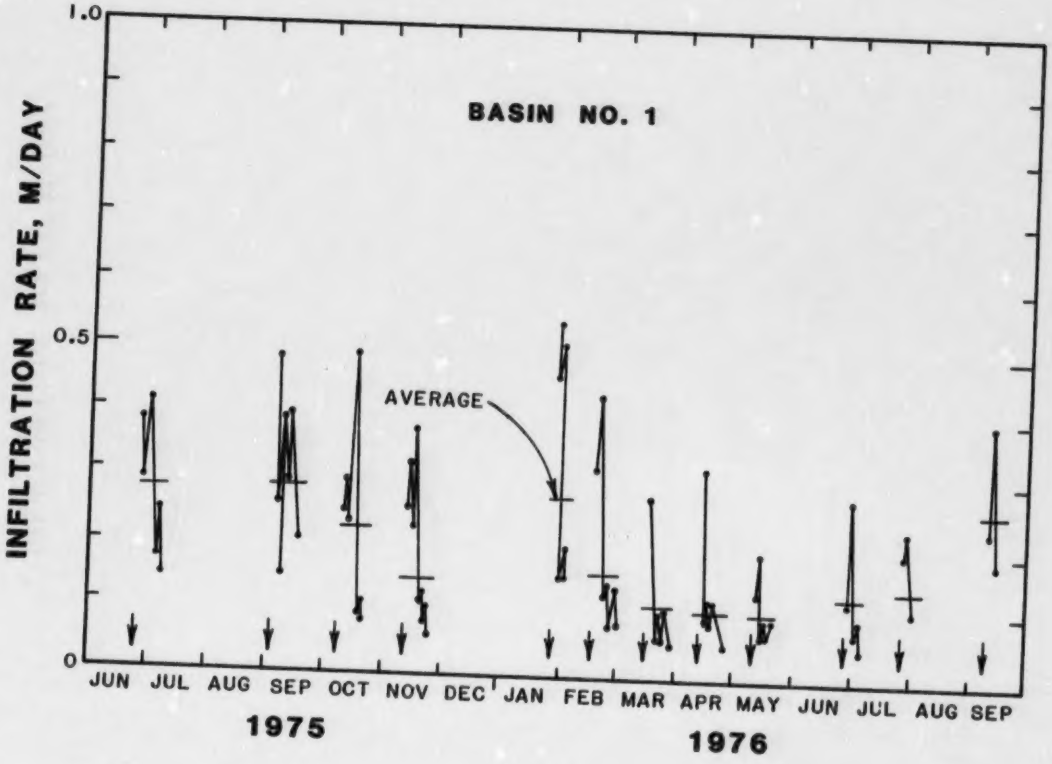

Figure 8. Pre-bypass-channel infiltration rates for basin 1. The downward arrows indicate start of flooding periods.

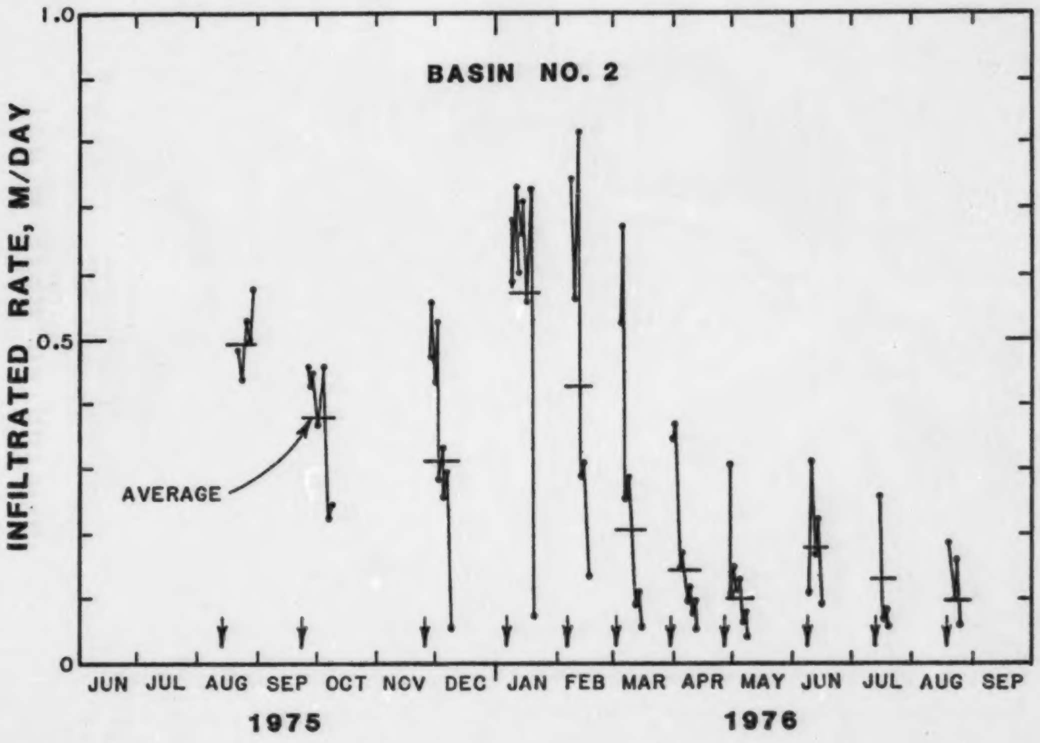

Figure 9. Pre-bypass-channel infiltration rates for basin 2. The downward arrows indicate start of flooding periods.

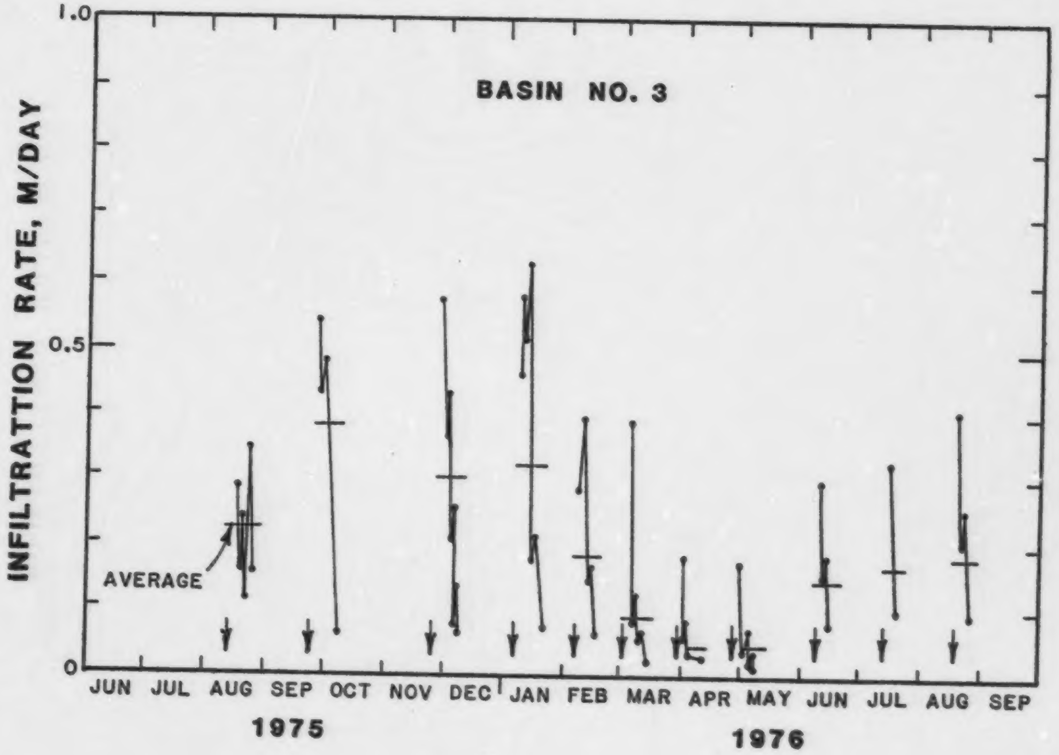

Figure 10. Pre-bypass-channel infiltration rates for basin 3. The downward arrows indicate start of flooding periods.

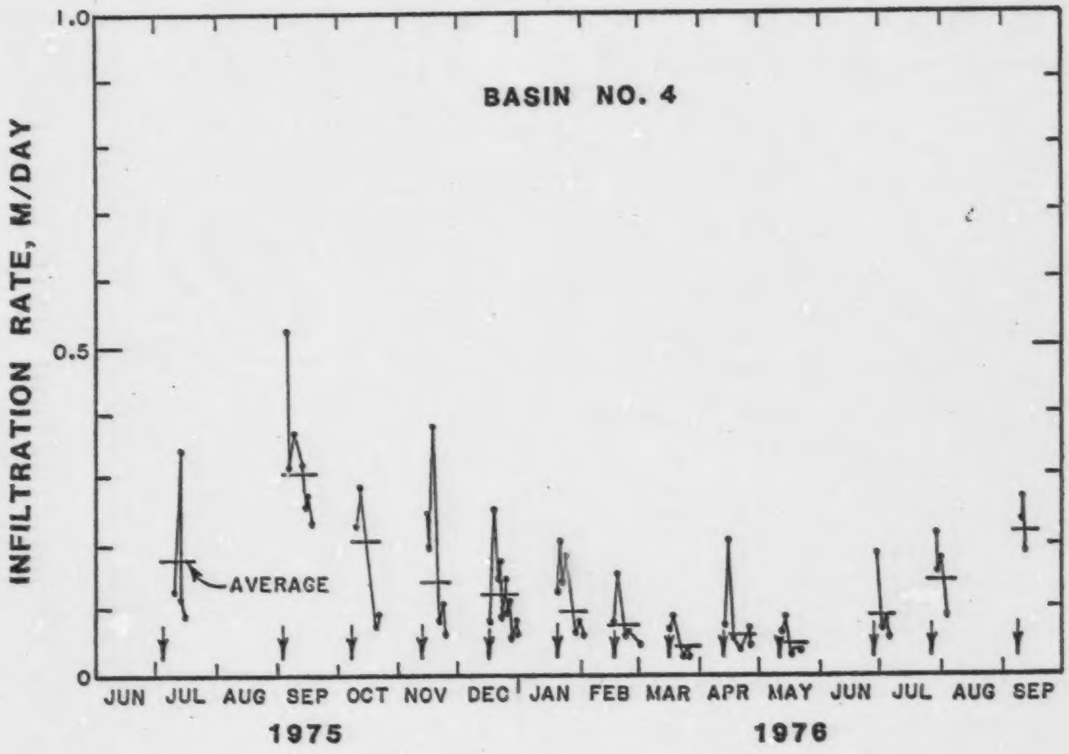

Figure 11. Pre-bypass-channel infiltration rates for basin 4. The downward arrows indicate start of flooding periods.

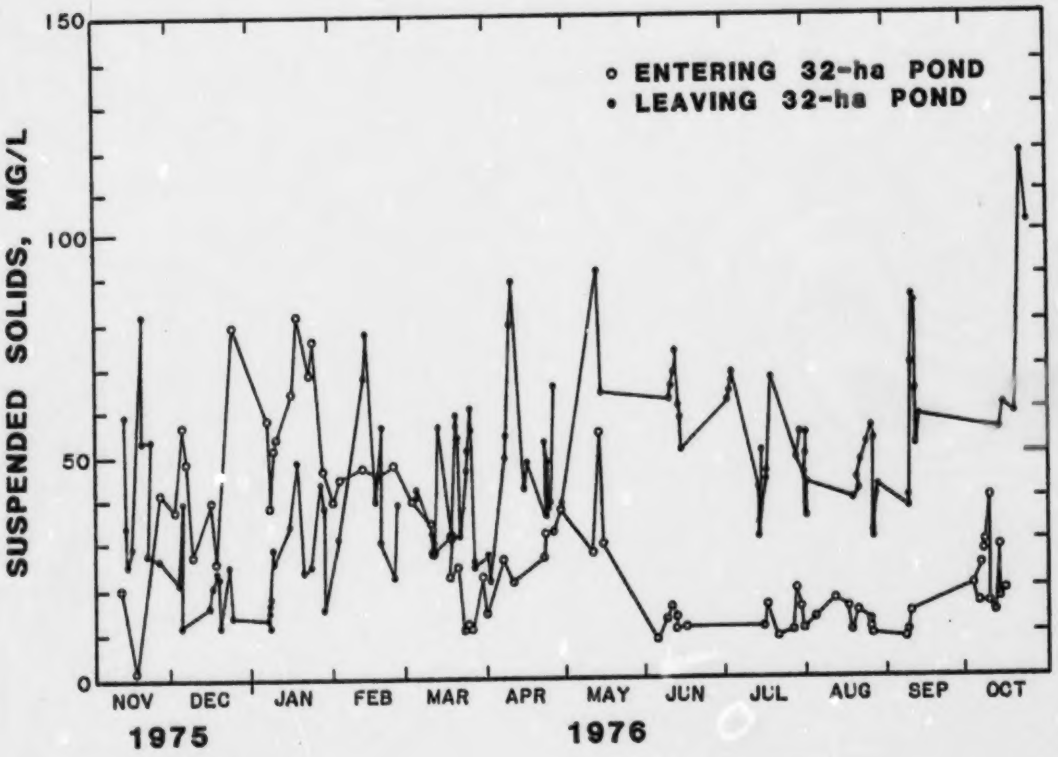

Figure 12. Suspended solids content of secondary effluent entering and leaving 32-ha lagoon.

reduce infiltration periods to one week and to allow drying periods of four weeks. Regular infiltration periods were stopped in December 1976 to enable some plot-type infiltration studies in the basins to evaluate effects of clogging and air-pressure buildup in the vadose zone on infiltration rates (see next section). The total infiltration or hydraulic loading for the 15-month period July 1975--September 1976 was:

Basin 1	24.4 m
Basin 2	38.4 m
Basin 3	25.6 m
Basin 4	20.1 m

Based on these data, the average annual hydraulic loading was 21.6 m. This is much less than would have been expected on the basis of the results of the Flushing Meadows Project (Bouwer et al., 1980), which produced hydraulic loading rates of 60 to 120 m/year on similar soil.

As discussed in the following section, the main reason for the low infiltration rates was the high algae content of the effluent after it had passed through the 32-ha lagoon. Algae accounted for almost all of the suspended solids in this effluent (Figure 12). The algae not only accumulated on the bottom of the infiltration basins where they formed a "filter cake" with high hydraulic impedance, they also caused precipitation of calcium carbonate and resulting cementing of the upper soil layer. Precipitation of calcium carbonate was due to the uptake of carbon dioxide from the water as the algae photosynthesized, which increased the pH of the water to 9 and sometimes even more. Algae identification in 1979 showed that the main algae bloom organism was Carteria klebsii Dillworth (see section Basin Infiltration Rates With Secondary Effluent From Bypass Channel).

Another effect of the low infiltration rates was that water continued to stand for very long times in low spots of the basins, after most of the water had drained through the outlet structures to start a drying period. Water would stand in the low spots for more than a week and sometimes for the entire drying period, especially in basin 4 which drained poorly and often had water standing in it for more than one-half the bottom area at the end of the drying period. Algae continued to flourish in these pools of stagnant water, causing further reduction in infiltration rate. The poor recovery in infiltration rates for the low spots with standing water reduced the area of the infiltration basins where infiltration could occur at normal rates. Alternatively, very long drying periods were required if complete drying and adequate infiltration recovery also had to be obtained for the low spots. Both effects reduced the hydraulic loading rates of the basins.

EFFECTS OF AIR PRESSURE BUILDUP AND SUSPENDED SOLIDS ON INFILTRATION

Before definitely deciding that a channel should be constructed around the 32-ha lagoon to convey secondary effluent directly to the infiltration basins, and, hence, to avoid growth of algae in the effluent, studies had to be made to make sure that algae were the only cause of the low infiltration rates and that there were no other reasons. Low infiltration rates, for example, could also be caused by increases in air pressure below the advancing wet front in the vadose zone. With the long narrow infiltration basins of the Flushing Meadows Project, this was not a problem because the air could readily escape laterally. Below the large infiltration basins of the 23rd Avenue Project, however, distances for lateral air movement were much greater and air pressures could indeed increase. If this were true, a bypass channel would not do much good. Rather, the infiltration basins would then have to be split into smaller units and/or they would have to be equipped with air-pressure relief pipes. To investigate this possibility, detailed studies on air pressures in the vadose zone were performed in 1976. In addition, infiltration tests were carried out on selected areas with effluents of different suspended solids and algae contents to determine how much increase in infiltration could be obtained by decreasing the algae content.

Air pressures in the vadose zone were measured with a cluster of three piezometers at depths of 3 m, 6.1 m, and 12.2 m (Figure 13). The piezometers were installed in a vertical hole drilled in the center of basin 2 with a 0.6-m diameter rotary-flight auger. Each piezometer consisted of a 0.3-m section of 7.5-cm diameter, perforated pipe at the bottom that was connected to the surface with 1.2-cm diameter plastic pipe. Each perforated section was located in a 0.5-m thick gravel layer. Grout was placed between the gravel layers in the hole to prevent contact between the piezometers. The piezometers were connected with 0.6-cm polyethylene tubing to a transducer-recording system on the center dike for monitoring air pressures.

Of the three piezometers, only the one at a depth of 12.2 m showed air-pressure increases when a new flooding period was started. The air pressure increased immediately after infiltration began, reached a peak after about five days and decreased to zero after about nine days (Figure 14). The peak value of the air pressure varied essentially linearly with the initial infiltration rate in the basin (Figure 15). Since the infiltration rate continued to decrease during flooding and did not show a rebound after air pressures declined to zero (Figure 9), soil clogging and not air pressure build-up appeared to be the major cause of the decreasing infiltration rates.

To further ascertain that air-pressure build-up was not the cause of low infiltration rates, infiltration rates were measured in 1977 on two square areas of 1 m by 1 m, obtained by installing sheetmetal borders in the soil of basin 2, near the Center Well. The square areas were flooded with water free from suspended solids (renovated sewage water from the Center Well was used for this purpose), to a depth of 15 cm. This depth was then maintained with a float valve. The rest of the basin around the square areas was initially kept dry. The infiltration rate during this period was essentially constant at about 0.75 m/day (Figure 16). Since the basin around the square areas was kept dry during this measurement, there was no air-pressure buildup in the vadose zone below

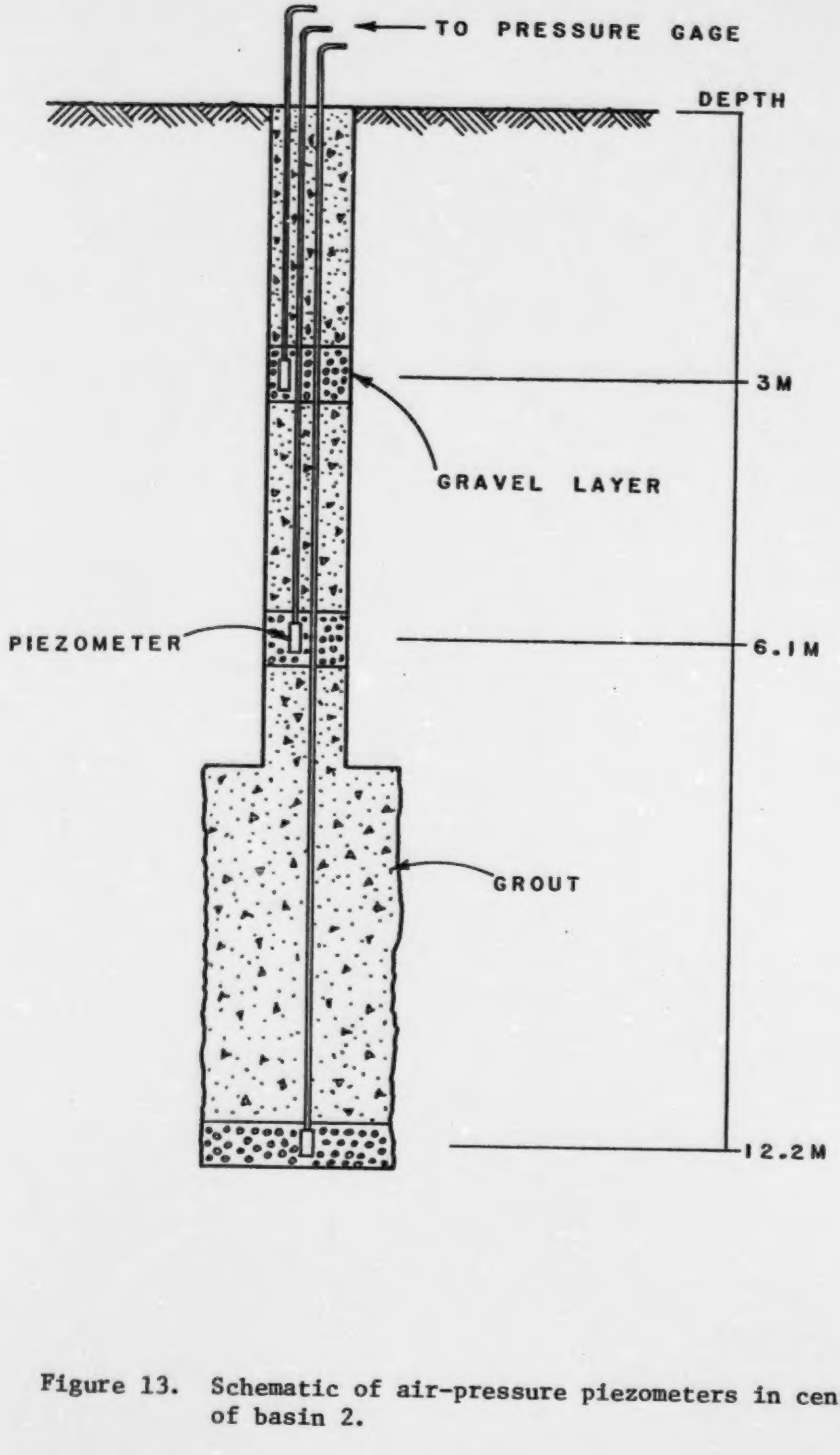
TO PRESSURE GAGE
DEPTH
3M
GRAVEL LAYER
PIEZOMETER
6.1M
GROUT
12.2M

Figure 13. Schematic of air-pressure piezometers in center of basin 2.

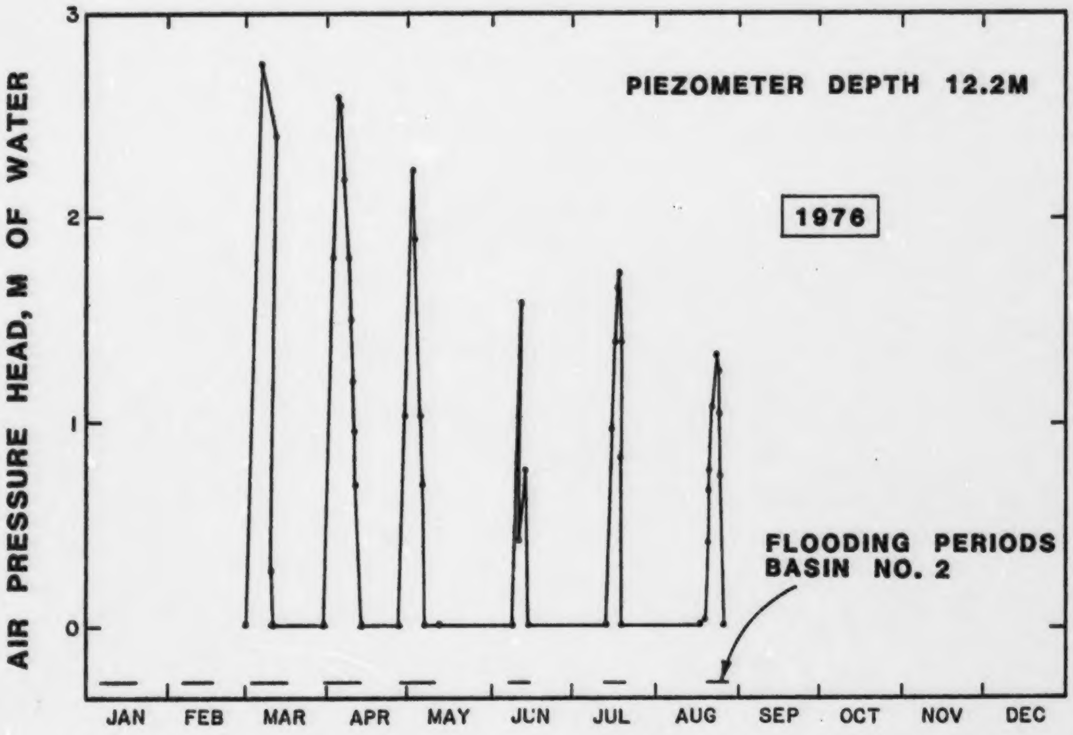

Figure 14. Pressure heads of air (in m of water) at 12.2-m depth in center of basin 2.

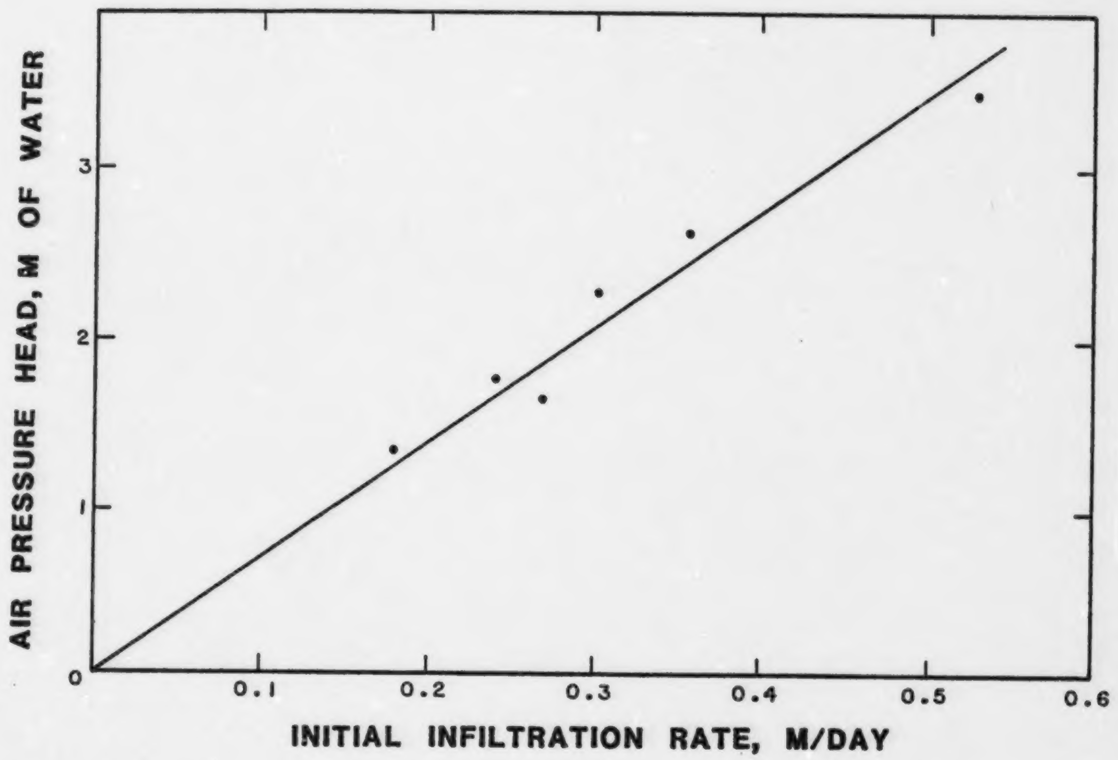

Figure 15. Relation between peak air-pressure heads at 12.2-m depth and initial infiltration rate.

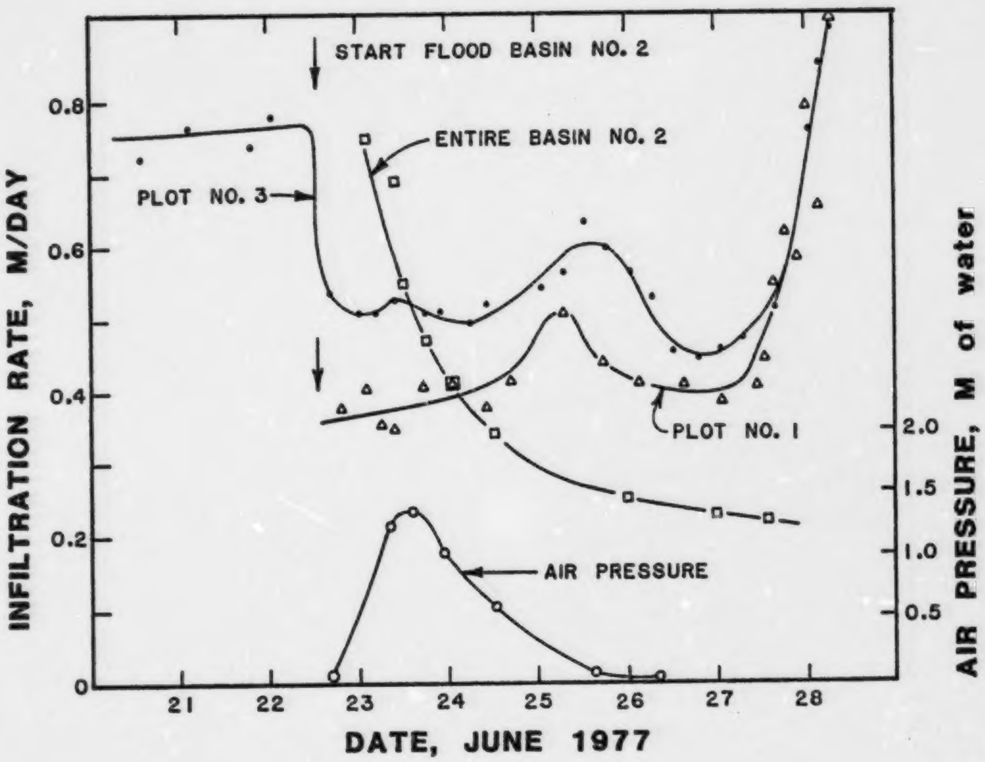

Figure 16. Infiltration rates of square-meter areas before, during, and after flooding entire basin, and air pressures at 12.2-m depth.

the wetting front. If the rest of the basin were flooded, however, there would be an increase in air pressure in the vadose zone, which in turn would decrease the infiltration rate of the square areas. Flooding the rest of the basin would also reduce the infiltration rate from the square areas because divergence of the flow due to edge effects would be eliminated.

Previous studies with an electrical resistance network analog (Bouwer, 1960) have indicated that for the soils of the infiltration basins which have an air-entry value of -38 cm water as determined with the air-entry permeameter (Bouwer, 1966), an infiltration area of 1 m^2 will undergo a reduction in infiltration rate of about 50 percent when the area is surrounded by water and flow divergence can no longer take place. Thus, if the infiltration rate in the 1 m^2 areas in basin 2 would be reduced by more than a factor of two due to flooding the entire basin, air-pressure build-up in the vadose zone would also have an effect on infiltration. The results (Figure 16), however, showed that the infiltration rate was reduced only from 0.75 m/day to 0.50 m/day, or not quite by a factor of two. Thus, the air-pressure build-up in the vadose zone, which was about 1.4 m water at 12.2-m depth (Figure 16), had no effect on the infiltration rate from the square areas. The infiltration rate for the entire basin was 0.73 m/day at the beginning of flooding and 0.15 m/day after five days. The increase in infiltration rate from the square areas (Plot No. 3) on about 25 June probably was caused by a temporary drop in water level in the rest of the basin, which produced some divergence of the flow from the square areas. When flooding the entire basin was stopped on 27 June, the infiltration rate from the square areas began to increase (Figure 16). This was probably due to the decrease in (negative) pressure heads of the water in the upper portion of the soil profile as the profile drained toward the groundwater table at a depth of 21 m. The resulting pressure heads were much more negative than normally occurring in a wetted zone with downward moving water, thus producing higher infiltration gradients.

The effect of suspended solids content of the water on infiltration rate was studied in the fall of 1976, using three plots of 1 m^2 each in basin 2. The plots were flooded with renovated water, secondary effluent as it entered the 32-ha lagoon, and secondary effluent as it left the 32-ha lagoon. The results (Table 1) indicated that the effluent from the 32-ha lagoon was the only water to produce a significant reduction in infiltration rate during the 7 to 12 days that the areas were flooded. Since the waters used in the experiment were stored in 55-gallon drums and metered from there into the plots, some settling of suspended solids occurred in the drums, as evidenced by the relatively low suspended solids content of the 32-ha pond effluent. Assuming that infiltration decreased linearly with accumulated suspended solids load on the bottom of the basin, a suspended solids content of 70 mg/ℓ of the 32-ha lagoon effluent (which is typical for summer months) would result in a final infiltration rate of only 0.12 m/day at the end of the 9-day flooding period. These results indicated that the use of secondary effluent directly from the treatment plant instead of the algae-laden effluent from the 32-ha pond for the infiltration basins should more than double the hydraulic loading rates (considering also the improved effects of higher infiltration rates on drying of the basins between flooding periods and infiltration recovery).

TABLE 1. INITIAL (I_o), FINAL (I_{end}), AND AVERAGE INFILTRATION RATES FOR VARIOUS WATERS IN 1-m^2 PLOTS IN BASIN 2, 1976

Plot	Water	Flooding period	Suspended solids (mg/ℓ)	I_o (m/day)	I_{end} (m/day)	$I_{average}$ (m/day)
1	Renovated	23-30 Sep	0	0.64	0.61	0.61
1	From 32-ha pond	13-22 Oct	35	0.46	0.34	0.40
2	Secondary effluent	4-16 Oct	9	0.82	0.70	0.73
3	Renovated	14-22 Sep	0	0.67	0.73	0.70
3	Secondary effluent	2-14 Nov	10	0.46	0.55	0.52

The results of the air-pressure and infiltration studies indicated that elimination of algal blooms in the effluent by constructing a bypass channel in the 32-ha pond to get treatment plant effluent directly to the infiltration basins was the appropriate solution to the problem of low infiltration rates in the basins. Construction of the bypass channel was not only delayed by the usual administrative procedures, but also by a failure to get a bid when the job was first advertized. After combining it with other construction work, and readvertizing, a bid was obtained early in 1979. Construction began in the spring of 1979 and the bypass channel was put into operation in June 1979. The channel was constructed by building a levee inside the 32-ha pond on the south side to create a channel from the last inflow culvert of the pond to the inlet structures of the infiltration basins (Figures 1 and 6).

BASIN INFILTRATION RATES WITH SECONDARY EFFLUENT FROM BYPASS CHANNEL

Because of above-average rainfall in the winters of 1977 and 1978, considerable vegetation had grown in the basins during the period of inactivity that started in September 1976. Most of the vegetation consisted of spring annuals, which dried in late spring and summer. However, there was also considerable growth of salt cedars (Tamarax gallica), particularly in basin 4. Salt cedars also developed in the east side of basin 3, and in a more scattered fashion in basin 1. Basin 2 had the fewest salt cedars. Many of the salt cedars were over 2 m in height at the end of 1979. While scattered growth of salt cedar is not objectionable, dense stands should be avoided because they shade the bottom and retard the drying and infiltration recovery process. Thus, most of the salt cedars were removed in the spring of 1980, before they got too large. Only a few salt cedars were left in each basin for scenic and wildlife reasons. The normal schedule of flooding and drying generally prevented vegetation from getting established. Once plants managed to grow above the water, however, they tended to thrive under the intermittent application of sewage effluent. The bottom condition of the basins when flooding was resumed in June 1979 thus was mostly bare soil with considerable dead vegetation and scattered salt cedars about 2.5 m in height.

The outflow weirs at the west end of the infiltration basins (Figure 1) were set to maintain an average water depth of about 0.25 m in the basins. This was considerably less than the 0.9 m maintained in the 1975-76 period. A smaller water depth was considered preferable, however, since it would reduce the surface detention time of the effluent water in the basins and, hence, the growth of algae. For example, if the infiltration rate is 0.3 m/day and the water depth is 0.3 m, the average turnover time of the effluent in the basin is one day. However, if the water depth is 0.9 m, the turnover time would be three days, assuming that the infiltration rate remains the same. This assumption is reasonably valid, since water depth has only a minor effect on infiltration rate after the wet front has sufficiently advanced and if there is no surface clogging of the soil.

Flooding and drying periods initially were about nine days each. Deviations of one or two days sometimes occurred due to weekends or to accommodate a study on N transformations in the soil (see Section 5, Nitrogen Transformations and Denitrification Rates). Infiltration rates for the first inundation period were so high (about 2 m/day) that only one basin could be flooded at a time. Thereafter, the basins were flooded in pairs, symmetrical with respect to the center dike. At the end of October 1979, the secondary effluent started to contain more fine suspended solids and assumed a greyish color. This caused more clogging of the soil in the infiltration basins, and longer drying periods were required for all the effluent to disappear into the ground. Thus, starting in November 1979, only one basin was flooded at a time and the sequence for each basin was one week flooding--three weeks drying. Infiltration rates for the basins, calculated from the difference between the inflow and outflow rates, are shown in Figures 17, 18, 19, and 20. The infiltration rates still showed considerable decrease during flooding, indicating that soil clogging still took place. However, since the algae content of the secondary effluent was much less than before construction of the bypass channel, the basins dried rapidly and there was good infiltration recovery. Average infiltration rates were on the order of 0.4 m/day for basins 1, 2, and 4, and 0.6 m/day for basin 3.

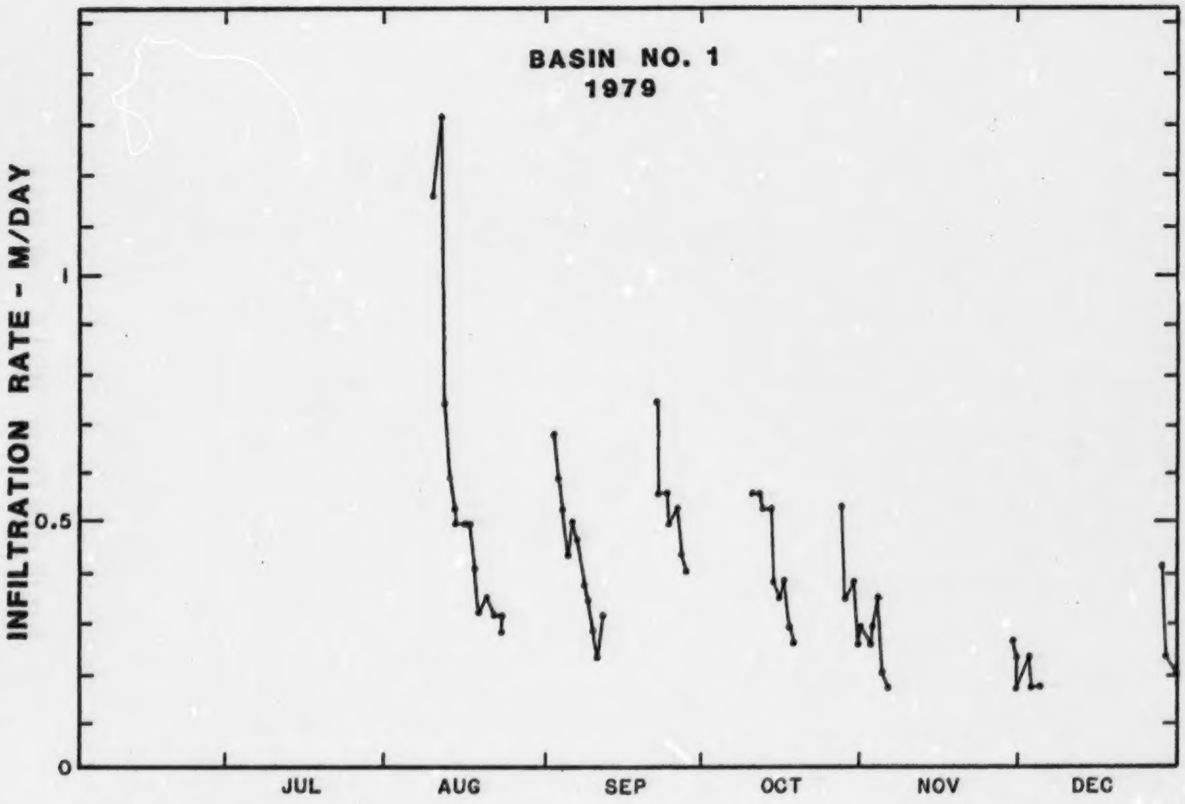

Figure 17. Post-bypass-channel infiltration rates in 1979 for basin 1.

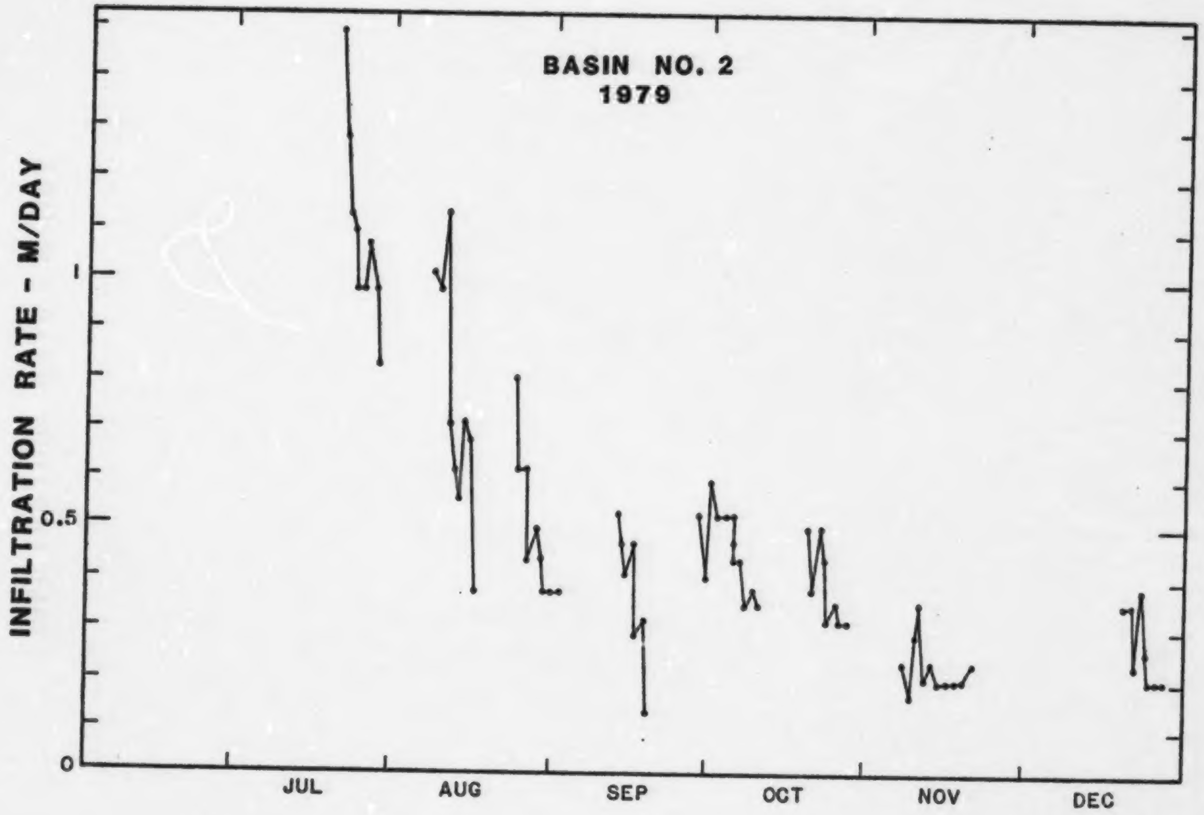

Figure 18. Post-bypass-channel infiltration rates in 1979 for basin 2.

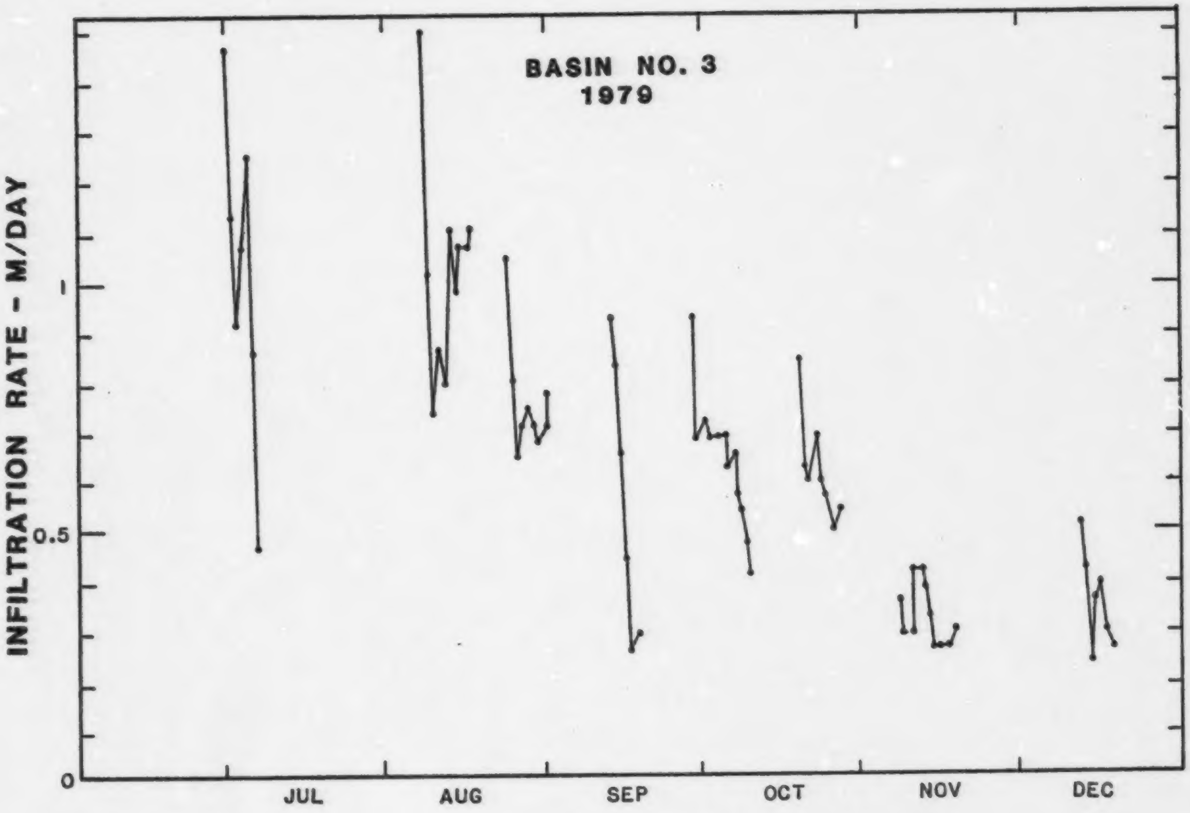

Figure 19. Post-bypass-channel infiltration rates in 1979 for basin 3.

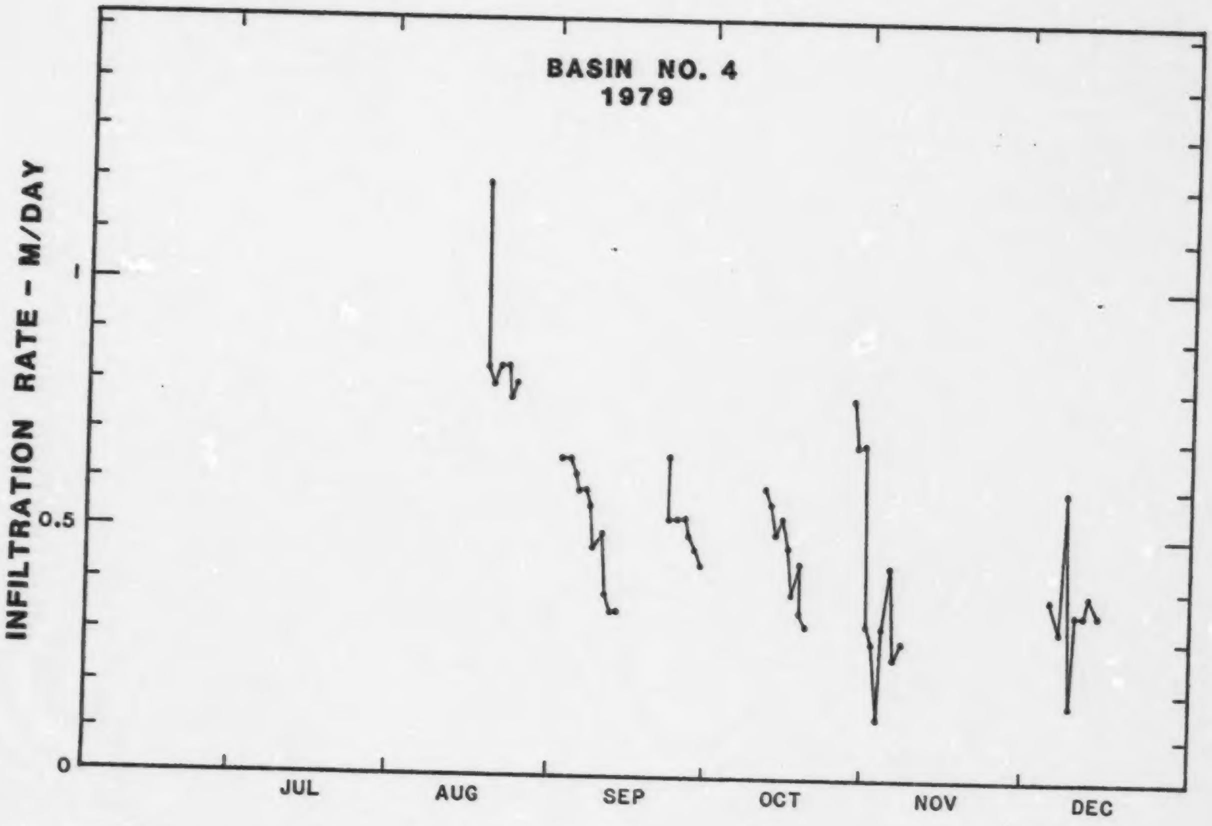

Figure 20. Post-bypass-channel infiltration rates in 1979 for basin 4.

Accumulated infiltration rates for 1979 are shown in Figure 21. All curves exhibited about the same slope, indicating that the basins all had about the same infiltration characteristics. The curves for basins 1 and 4 were below those for basins 2 and 3 because basins 1 and 4 got a later start. The hydraulic loading rate for one-half year was about 38 m, which would be about 76 m/year.

Infiltration rates in 1980 started out on the order of 0.3 m/day in January and February, using a schedule of one week of flooding and three weeks of drying (Figures 22, 23, 24, and 25). During March and the first half of April, the project was dry to allow cleaning of the bypass channel which had become clogged with tumbleweeds. Burning was not very effective, so that the tumbleweeds had to be removed with a grade-all.

After the long dry-up, flooding and drying periods were set at two weeks each and infiltration rates were higher than before, but the average was mostly below 0.5 m/day. This was still below the anticipated rate for the surface "soils" in the basins, indicating that surface clogging or crusting still persisted (possibly residual cementing by calcium carbonate deposits due to the algae-laden effluent in 1975 and 1976). To break up the surface layer, the soil in the basins was ripped with a road grader. Disking or cultivating was not possible because of the many large rocks and boulders in and on the soil. The ripping teeth on the grader were mounted about 20 cm apart and penetrated the soil to a depth of about 10 cm. Following this operation, infiltration rates were so high that first only one basin could be flooded at a time with the flow from the bypass channel. Even then, the inflow was not sufficient to cover the entire basin and infiltration rates had to be calculated on the basis of the estimated flooded area. In September 1980, the basins could be operated again in pairs and flooding and drying periods again were two weeks each. The average infiltration rates now were on the order of 1 m/day, except for basin 4, where they were about 0.5 m/day. The increase in infiltration rate is reflected in the steeper slope of the accumulated infiltration curves after July (Figure 26).

The flooding depth of 1 m initially used for the basins prevented the development of vegetation. However, when the depth was reduced to about 25 cm, a lush vegetation developed. Main plant species were barnyard grass (Echinochloa Crusgalli) and willow leaf (Polygonum Lapathifolium). The plants were about 0.5 to 1.5 m tall and since they grew well above the water, they provided ideal circumstances for mosquitoes to breed. To minimize mosquito breeding, the vegetation should be periodically mowed and water depths should be large enough to completely submerge the plants during flooding.

Extrapolation of the curves in Figure 26 indicates that the average hydraulic loading of the four basins would be about 85 m for the entire year. Of this amount, however, 21 m or only 25% infiltrated in the first six months of the year. Thus, the ripping, which cost $870 for the four basins, just about doubled projected annual loadings (from about 60 m to 120 m). Assuming that the basins would have to be ripped every year to maintain the high infiltration rates, the cost of increasing the

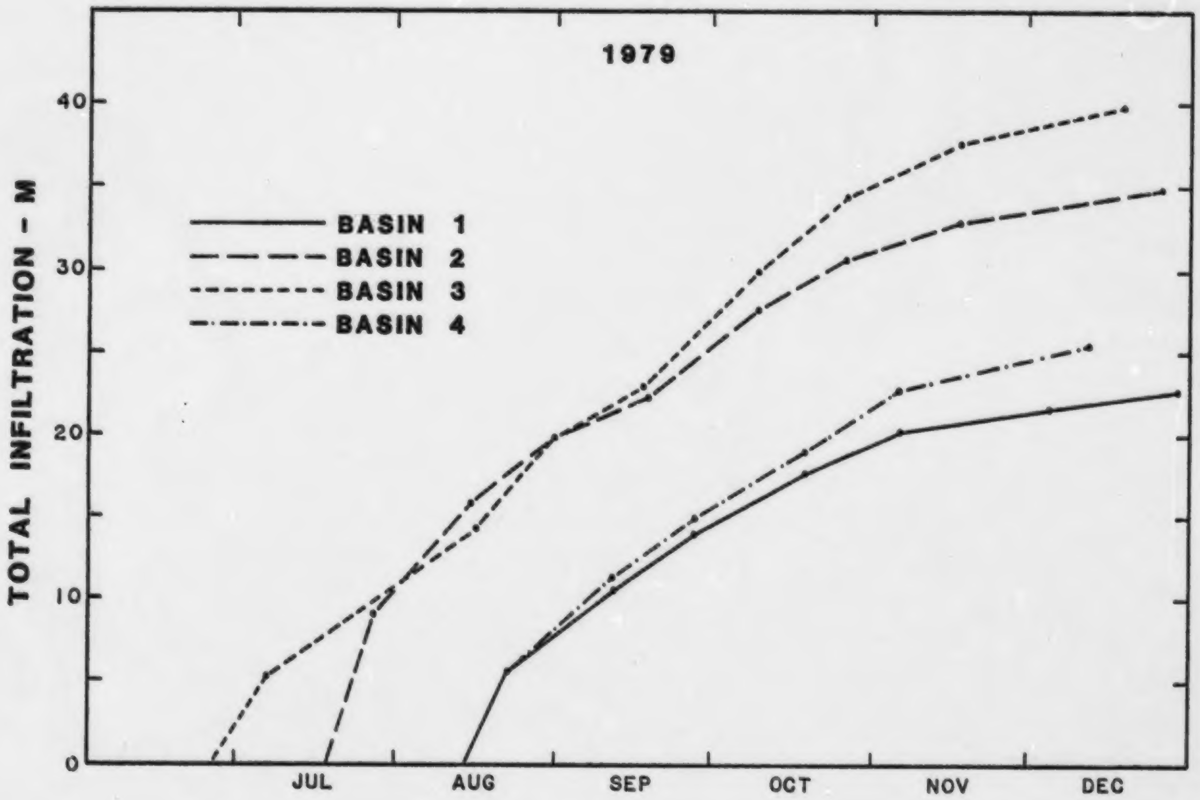

Figure 21. Accumulated infiltrations for 1979.

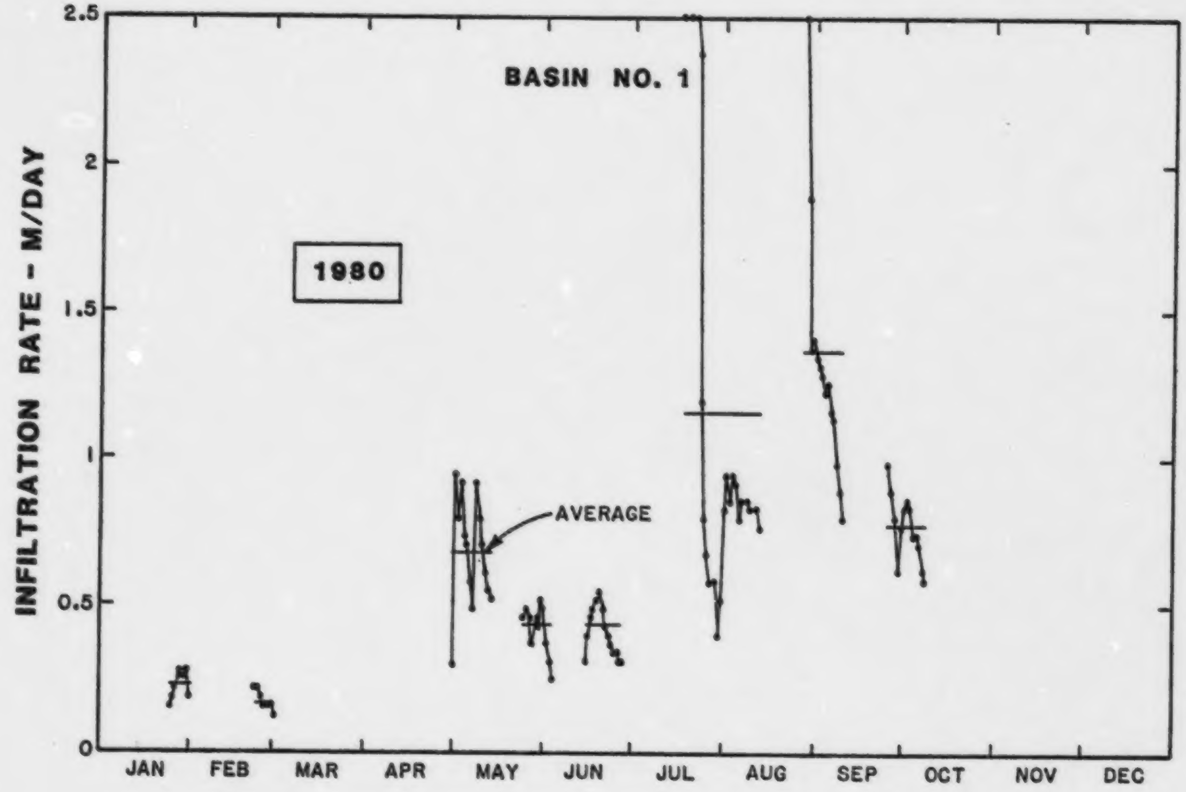

Figure 22. Post-bypass-channel infiltration rates in 1980 for basin 1.

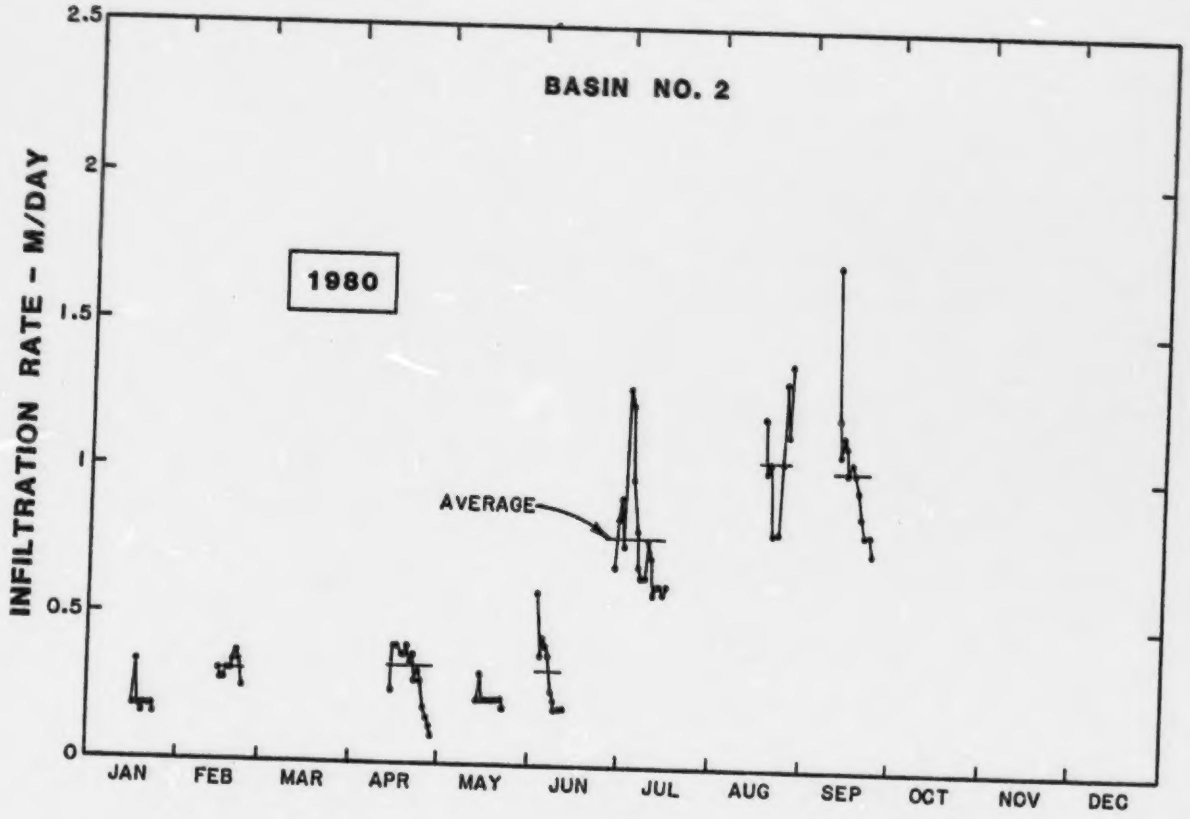

Figure 23. Post-bypass-channel infiltration rates in 1980 for basin 2.

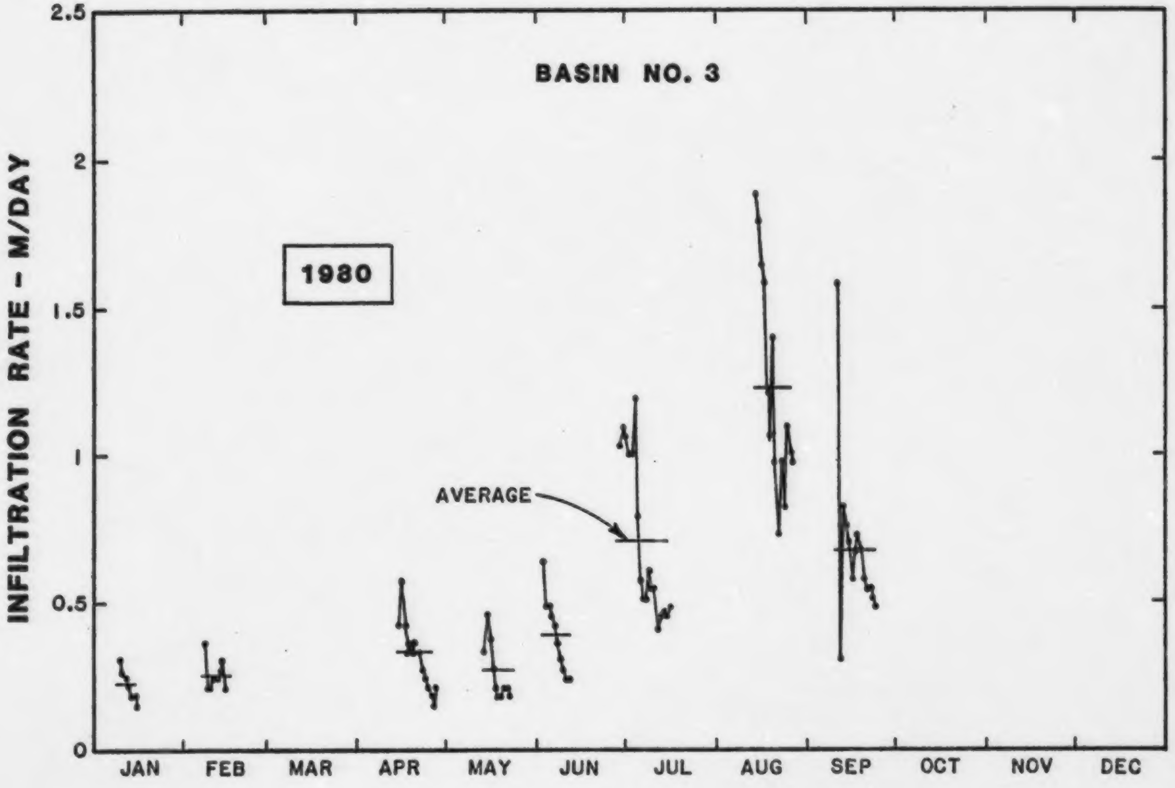

Figure 24. Post-bypass-channel infiltration rates in 1980 for basin 3.

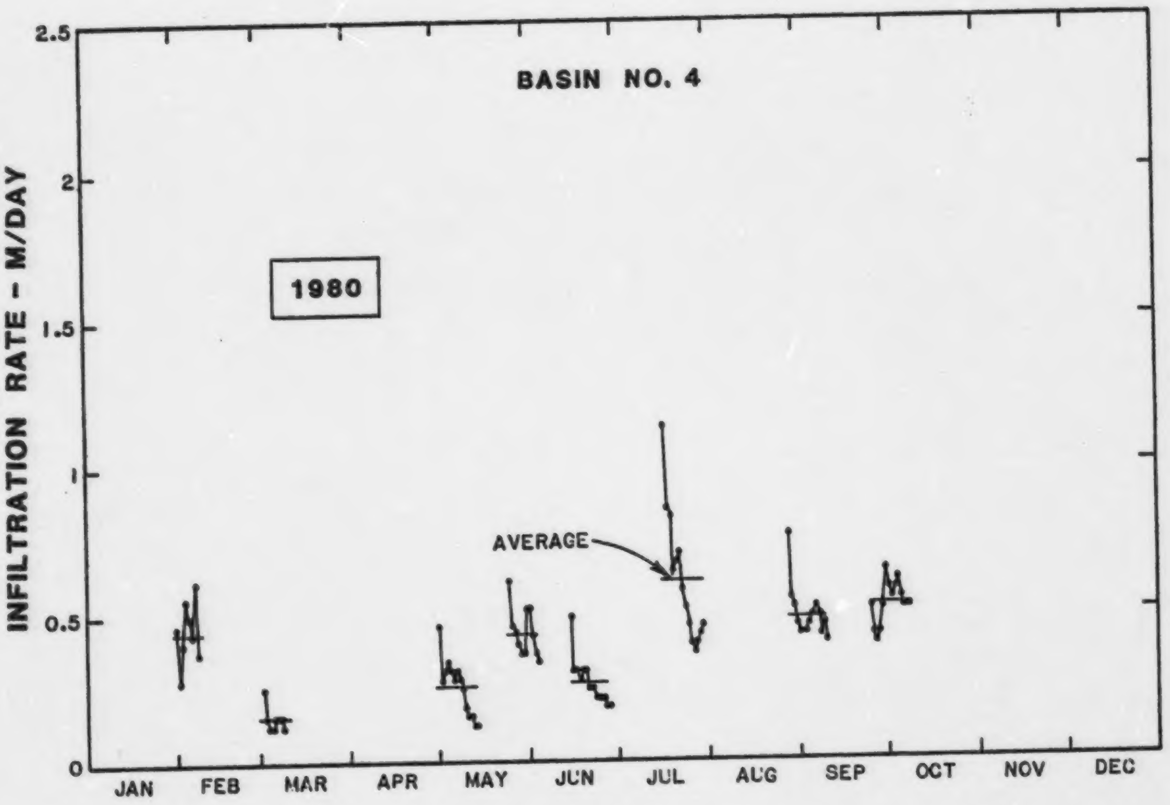

Figure 25. Post-bypass-channel infiltration rates in 1980 for basin 4.

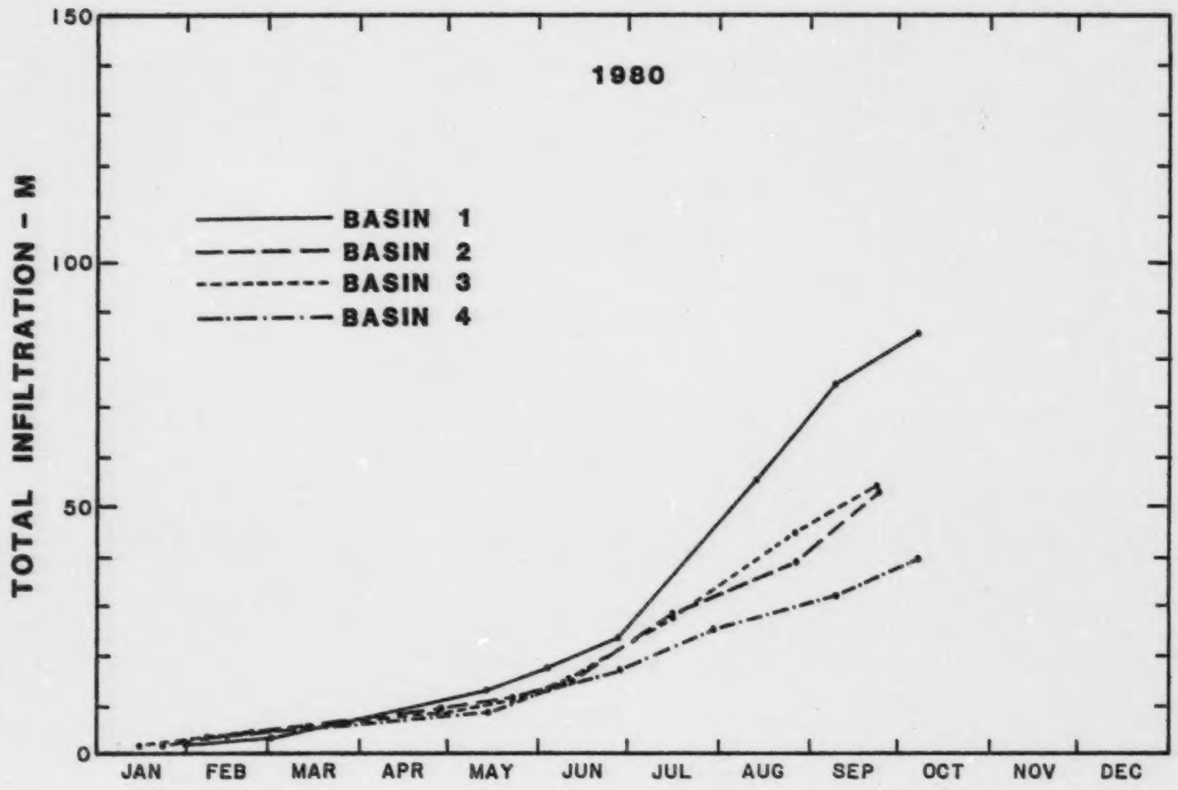

Figure 26. Accumulated infiltrations for 1980.

hydraulic loading rate by 60 m/year amounts to 870/60 x 16 x 10000 = \$0.00009/m^3 or \$0.09/1000 m^3 (\$0.11 per acre-foot) of extra water infiltrated. This indeed is an inexpensive way of increasing the capacity of the system, considering that the total cost of putting the effluent into the ground and pumping it up as renovated water is on the order of \$10 per 1000 m^3 or acre-foot.

The infiltration data indicated that the design hydraulic loading rate can be conservatively estimated at 90 m/yr. This would give the 16-ha rapid-infiltration system a capacity of 14.4 million m^3 per year (12,000 acre-feet/year) or almost 40,000 m^3/day (32 acre-feet/day, 11 mgd, or 7600 gpm). Additional hydraulic loading can probably be obtained by increasing the water depth in the infiltration basins in the winter. Due to the lower temperatures and reduced sunlight intensity, algal blooms would then be unlikely and the larger depths could increase infiltration rates. Small water depths should be maintained in the rest of the year, however. To pump renovated water from the aquifer at the design hydraulic loading rate would require three wells on the center dike (including, and similar to, the Center Well already installed). The additional wells should be somewhat deeper than the Center Well (100 m, for example) to maintain sufficient capacity when groundwater levels are low.

A hydraulic loading rate of 90 m per year would be about 14% of the average vertical hydraulic conductivity of the soil in the basins, assuming that the infiltration rate of approximately 1.8 m/day obtained when the basins were first flooded and the effluent advanced as sheet flow over the soil was a reasonable reflection of the vertical hydraulic conductivity of the upper soil layers.

The design hydraulic loading rate of 90 m per year is about four times higher than the rate of 22 m per year obtained in the period 1975-1976. The difference, of course, is due to the greatly reduced suspended solids (algae) content of the effluent going into the infiltration basins following the completion of the bypass channel. The suspended solids content of the effluent entering the infiltration basins through the bypass channel generally was below 10 mg/ℓ, and that of the effluent leaving the basins was often even lower (see Section 8, Suspended Solids). This is much less than the suspended solids contents of the basin inflows before construction of the bypass channel, which were on the order of 10 to 20 mg/ℓ in the winter and 50 to 100 mg/ℓ in the summer. Algae still grew in the infiltration basins, but they were more of the types that grow on the bottom or float on the surface, rather than the unicellular, suspended algae that previously gave the water such a green appearance. Thus, the effluent water itself in the infiltration basins remained quite clear.

Samples of floating and bottom algae in the infiltration basins and of the clear effluent itself were taken in July and November of 1979 for algae identification. This work was done by Mr. Andrew J. Lampkin III of the Department of Botany and Microbiology of Arizona State University. The main algae types in the bright green layer floating on the water were *Chlamydomonas* and *Pandorina*. These algae are vegetatively motile. The surface layer consisted mainly of non-motile resting stages (zygotes or aplanospores). *Oscillatoria* appeared to be the dominant mat-forming blue-green algae on the bottom of the basins. They were also noted in other samples (surface and plankton), as could be expected from the kinetics of mat formation and subsequent breakup. The main suspended or planktonic algae species was *Carteria klebsii*. This is the dominant bloom organism that probably caused the effluent to be so green and high in suspended solids when it went through the 32-ha pond before the bypass channel was constructed. A complete list of the algae in the various samples is presented in Table 2.

TABLE 2. RESULTS OF ALGAE IDENTIFICATION IN SAMPLES FROM INFILTRATION BASINS

	July	November
INFLOW (EAST END)		
Surface	*Ocsillatoria nigra* Vaucher *O. limnetica* Lemmerman *Chalamydomonas snowii* Printz *Oocystis parva* West & West Unidentified (29μ diameter) sphaeroid cell *Navicula* sp. #1 *Navicula* sp. #2	
Plankton	*Oscillatoria nigra* Vaucher *O. ornata* Kuetzing *Oocystis parva* West & West *Franceia ovalis* Lemmerman *Scenedesmus quadricauda* Smith *Marssoniella elegans* Lemmerman *Nephrocytium agardhianum* Naegeli *Scenedesmus incrassatulus* Bohlin *Chlorella* sp. *Sphaerocystis schroeteri* Chordat *Pandorina morum* Bory *Spondylomorum quarternarium* Ehrenberg *Stigeoclonium nanum* Kuetzing *Navicula* sp. #1 *Navicula* sp. #3 *Cryptomonas* sp. *Pandorina morum* Bory * *Carteria klebsii* Dillworth *	

(continued)

TABLE 2 (continued)

	July	November
Bottom	*Oscillatoria nigra* Vaucher *O. limnetica* Lemmerman *Navicula* sp. #1	
OUTFLOW (WEST END)		
Surface	*Oscillatoria limnetica* Lemmerman *Chlamydomonas snowii* Printz *Pandorina morum* Bory	*Carteria klebsii* Dillworth *Pandorina morum* Bory *Navicula* sp. #1
Surface (cont)	*Scenedesmus opoliensis* Richter *Chlorella* sp. *Marssoniella elegans* Lemmerman *Oocystis parva* West & West *Stigeoclonium* sp., germling *Navicula* sp. #1 *Navicula* sp. #2 *Nitzschia* sp. #1	
Plankton	*Oscillatoria nigra* Vaucher *O. limnetica* Lemmerman *Oedogonium* sp. *Pandorina morum* Bory *Navicula* sp. #1 *Nitzschia* sp. #1	*Scenedesmus incrassatulus* Bohlin *Scenedesmus quadricauda* Smith *Merismopaedia* (*Agmenellum*) *tenuissima* *Pandorina morum* Bory *Nitzschia* sp. #1 *Oscillatoria nigra* Vaucher *Zygnema* sp.
Bottom	*Oscillatoria nigra* Vaucher *O. limnetica* Lemmerman *Pandorina morum* Bory *Sphaerocystis schroeteri* Chordat *Gloeocystis ampla* Lagerheim *Chlorella* sp. *Navicula* sp. #1 *Navicula* sp. #2	*Oscillatoria limnetica* Lemmerman *Oocystis parva* West & West *Scenedesmus incrassatulus* Bohlin *Nitzschia* sp. #1

* Main species in stagnant effluent at end of bypass channel with algae bloom. *Carteria* was the main bloom organism.

SECTION 6

GROUNDWATER LEVELS

Elevations in m above sea level of the various wells (tops of casings and ground levels) and the average elevation of the bottoms of the infiltration basins are shown in Table 3. Ground levels at the wells were about 1.8 m above the basin bottom for the wells on the center dike, and 2.2 m for the wells on the levees north and south of the basins. All wells (including the Center Well) were pumped only for water sampling. Thus, groundwater levels in the area were not measurably affected by the pumping of the project wells.

From October 1975 to February 1976, groundwater elevations in the North Well ranged between 296 and 299 m above sea level, or between 15 and 18 m below the bottom of the basins (Figure 27). There was no clear response of the water level to infiltration from the basins. In February 1976, the water level started to decline, probably due to increased pumping of irrigation wells north of the project.

The water level in the Center Well initially was about the same as that in the North Well. In the beginning of April 1976, the measured groundwater levels in the Center Well suddenly dropped and remained about 3 m lower than those in the North Well for the rest of the measurements in 1976. Since this change seemed unrealistic from a hydraulic standpoint, it was attributed to measurement error with the bubble tube and the 100 psi pressure gage used for water level measurements in the Center Well (an electric conductivity probe was used for all other wells). Thus, when groundwater table measurements were resumed in 1979, a hole was drilled through the pump base of the Center Well, so that water levels could also be measured with the electric probe. This showed that the bubble tube measurements were lower by a distance of 4.6 m. Thus, all subsequent water level measurements in the Center Well were done with the electrical conductivity probe, and those after 1 April in Figure 27 were retroactively corrected.

The groundwater level in the Center Well continued to decline in 1976, most likely due to general groundwater pumping in the area. In September 1977, the water level in the Center Well was at a depth of 25 m. The water level in the North Well fell below the well bottom in May 1976. Water levels in the South Well showed a slow response, probably because silt and sand had accumulated on the bottom. Thus, water level measurements for this well are not shown. The South Well was drilled out and redeveloped in December 1979 and showed satisfactory response thereafter.

TABLE 3. ELEVATION OF WELLS AND BASIN BOTTOMS

Location	Place Surveyed	Elevation, m. above sea level	Height of top of casing above ground, m.	Sampling depth, m.
North Well (NW)	Top of cover plate	316.51	0.24	23
Center Well (CW)	Top of concrete base, east side	316.25	0.40	30-55
Center Well (CW)	Metal base at probe hole	316.40		
18-m Well (18W)	Top of casing, north side	316.66	0.88	18
24-m Well (24W)	Top of casing, north side	316.72	0.88	24
30-m Well (30W)	Top of casing, north side	316.72	0.88	30
South Well (SW)	Top of cover plate	316.69	0.49	23
Basin Bottoms (average)		314.00		

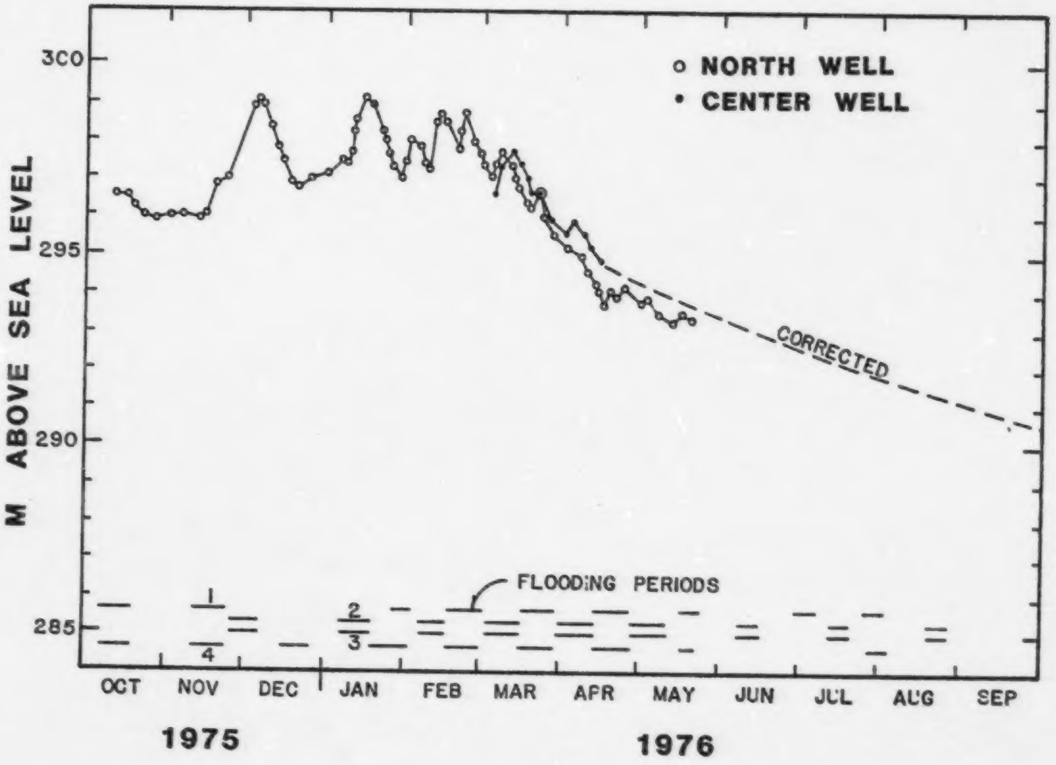

Figure 27. Water level elevations in the North Well and Center Well in 1975-1976. The dashed line indicates retroactively corrected measurements for the Center Well.

Groundwater level measurements were not made in the period 1977-1979, when the basins were not regularly flooded. An exception was the period March-July 1978, when a major flow in the Salt River and resulting seepage in the entire month of March caused a drastic rise in local groundwater levels. As shown in Figure 28, the water level in the North Well peaked out at a depth of about 12 m below the bottom of the basins and then rapidly declined after the river flow stopped and the groundwater mound due to seepage dissipated,and pumping of irrigation wells north of the project began.

In the spring of 1979, groundwater levels again were very high due to extended winter and spring flows in the Salt River (Figure 29). The water level in the North Well peaked at an elevation of 306 m above sea level, or a depth of 8 m below the bottom of the infiltration basins. The general decline of the water level in the North Well in the period April-July 1979 was most likely caused by dissipation of the groundwater mound formed by recharge from the Salt River bed and again by pumping of groundwater north of the project. The water level in the North Well rose when infiltration was first started (in basin 3), even though basin 3 was not the closest basin to the well. When basin 3 was no longer flooded, the water level in the North Well dropped, and then rose when basin 2 was flooded. The water level dropped again when basin 2 was dried and then it significantly rose in the beginning of July when basins 1, 2, and 3 were all flooded about the same time, etc. The general rise of the water level in the North Well, starting about the middle of July with the exception of a low in September (Figure 29), was probably due to recharge from the infiltration basins and to a reduction in groundwater pumping from irrigation wells north of the project later in the summer.

The water level in the North Well showed the typical response to normal diurnal changes in barometric pressure. During the day, heating of the air caused barometric pressures to decrease which in turn caused the water level in the well to rise. At night, the reverse took place. Maximum barometric pressures generally occurred in the morning (0800-1000) and were about 3 mm of mercury higher than the minimum pressures which generally occurred at the end of the afternoon (1700-1800) The effect of these fluctuations on water levels in the North Well are illustrated in Figure 30 for the period 26-28 May. Despite the fact that the water level was in a declining trend, slight water level rises occurred in the first few hours after noon.

The 1979 Center Well water levels were about 1.5 m higher than the water levels in the North Well, especially when basins 2 and 3 were flooded at the same time and groundwater mound formation in the center of the project was most pronounced. General fluctuation patterns for both wells were fairly well in phase (Figure 29).

Of the three new wells drilled with the air-rotary method in the center of the project in the first part of December 1979, water levels were highest in the 18-m Well and lowest in the 30-m Well (Figure 31). This indicated that the hydraulic head in the aquifer decreased with depth in the center of the project, particularly when basins 2 and 3 were

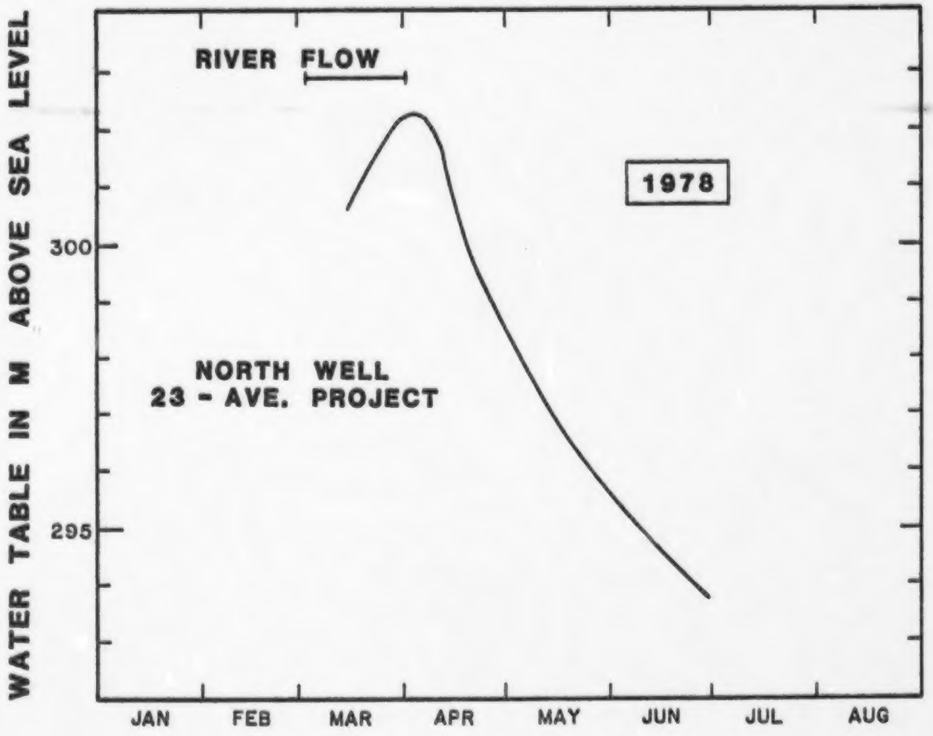

Figure 28. Water level elevations for the North Well showing effect of seepage from Salt River in March 1978

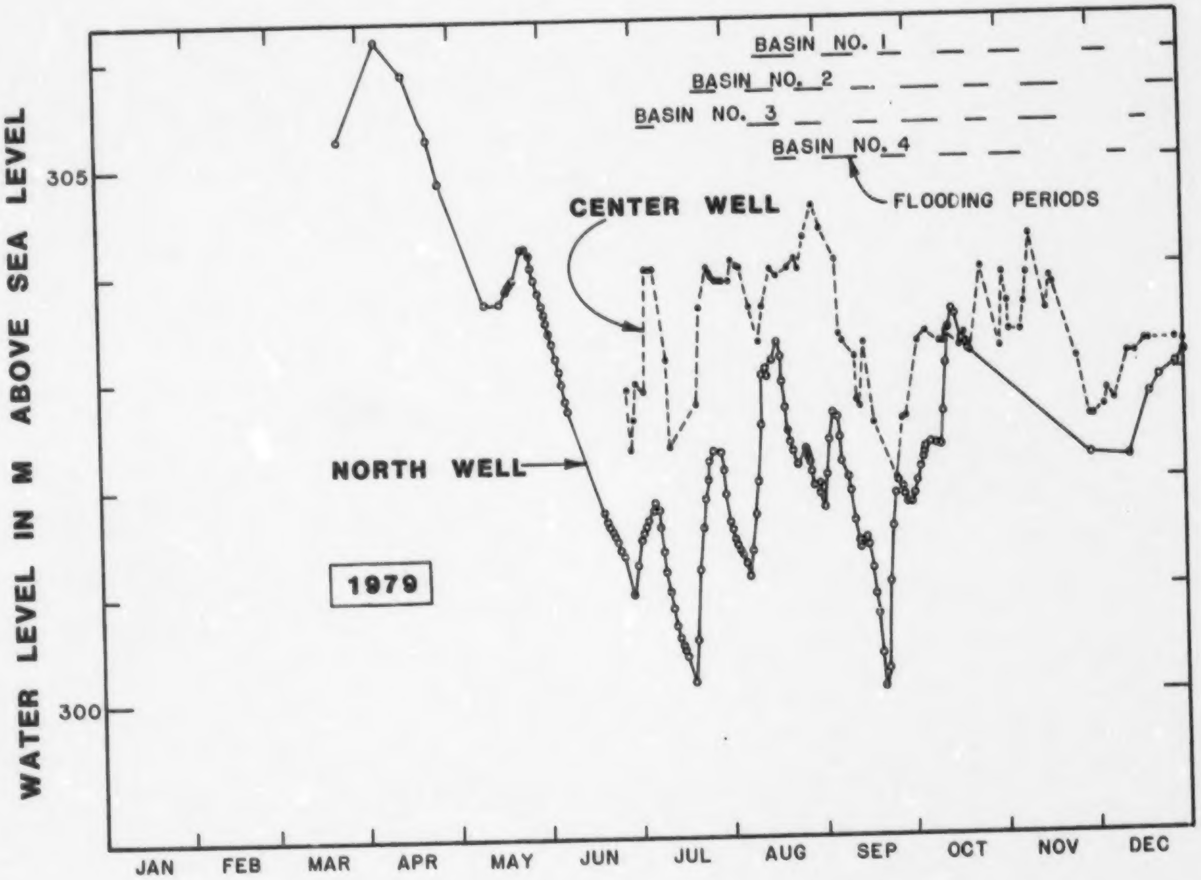

Figure 29. Water level elevations in North Well and Center Well in 1979.

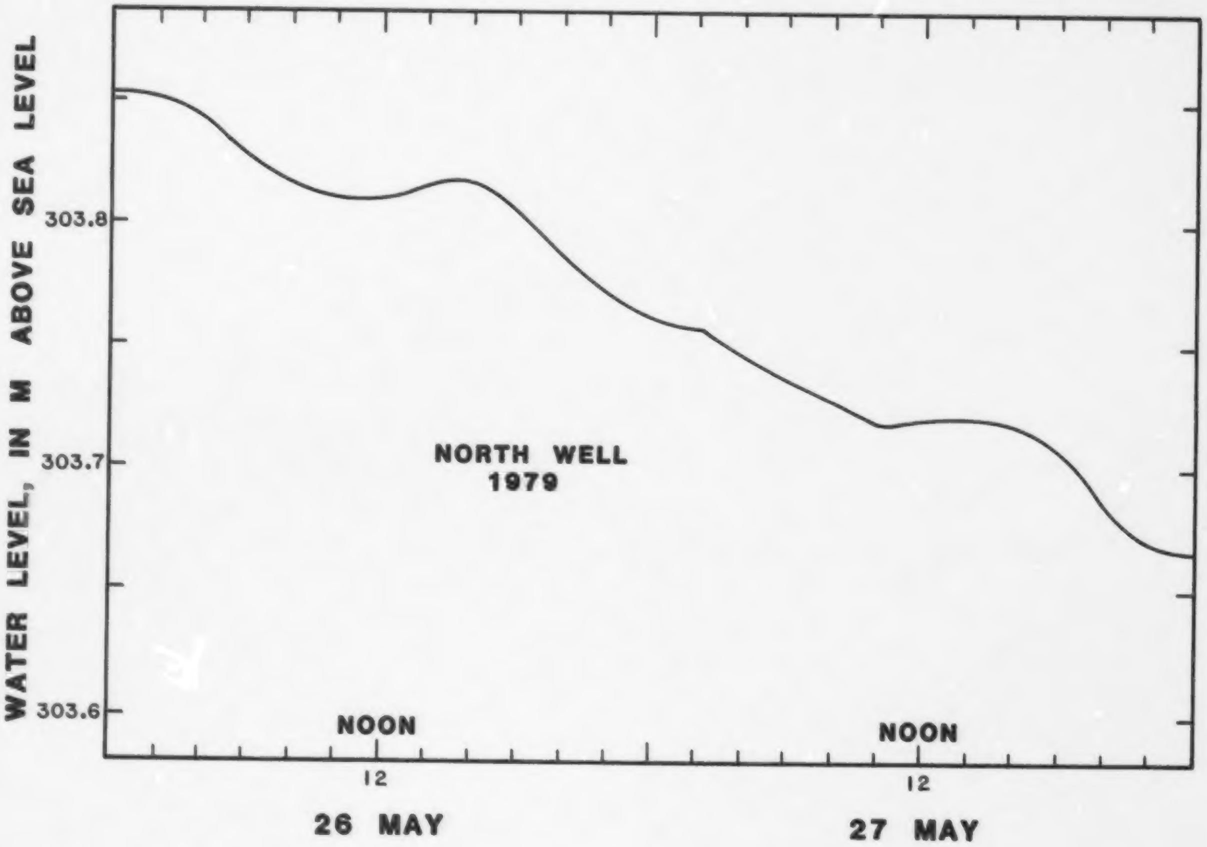

Figure 30. Diurnal water-level changes in North Well.

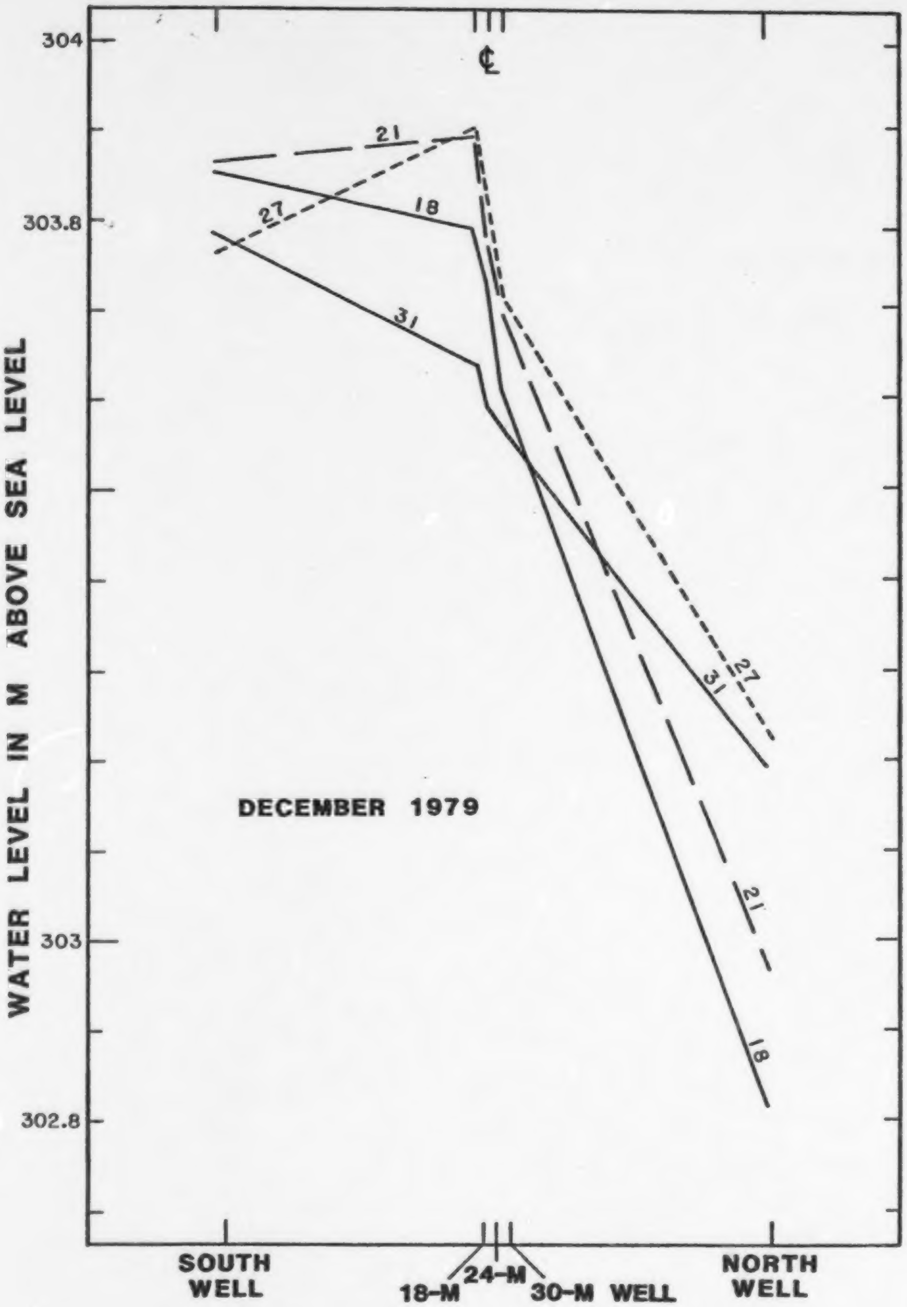

Figure 31. Groundwater level profiles from south to north midway across basins in December 1979 (numbers on curves indicate day of month).

flooded, and there was more downward movement in the center of the groundwater mound than when basins 1 and 4 were flooded. Thus, the water level elevations in the three center wells differed more on 18 December 1979, when basin 3 was just dried and basin 2 was just flooded than on 31 December 1979, when basin 1 was flooded only. As a whole, the water levels in the 18, 24, and 30-m Wells were about 0.5 m higher than those measured in the Center Well (Figures 29 and 31).

South to north groundwater level profiles, as obtained from measurements in the South Well, the 18-m Well, 24-m Well, 30-m Well, and North Well, are shown in Figure 31. The water levels of the 18-, 24-, and 30-m Wells should have been plotted on the centerline of Figure 31, but they were separated by 2.5 mm on the graph to better distinguish between the water levels in each well. Figure 31 shows that the groundwater gradient is predominantly from the south to the north. On 21 and 27 December, there was also flow to the south from the center of the project, as could be expected if the static groundwater table were essentially level and groundwater movement caused by the recharge from the infiltration basins were dominant. The southward flow from the center of the project could have been an aftereffect of the flooding of basins 3 and 4, and resulting groundwater mounding below the south half of the project.

The general slope of the groundwater table to the north continued in 1980, as evidenced by the water levels in the South, Center, and North Wells (Figure 32). The rise of these water levels in January and February 1980 was due to recharge from the Salt River, which for the third year in succession was flowing for the first few months of the year, this time at rates from 50 to 50,000 m^3/second. The decline of the groundwater levels from March to August 1980 was due to dissipation of the groundwater mound caused by recharge from the Salt River and to groundwater pumping from irrigation wells north of the project. In August, groundwater levels began to rise again, probably because of increased infiltration rates in the basins (see Figures 22, 23, 24, and 25) and of decreased groundwater pumping for irrigation. The curves in Figure 32 indicate that the depth of the groundwater table below the bottom of the recharge basins varied from a minimum of about 3 m on the south side of the project at the end of February to a maximum of about 12 m on the north side of the project in August.

Groundwater levels in the 18-, 24-, and 30-m Wells in the center of the project were very similar (Figure 33). As could be expected, the groundwater levels in the 18-m Well were highest, and also above the water levels in the Center Well, which is perforated from 30 to 55 m and gives the average hydraulic head for that depth range. These differences are due to downward flow components in the center of the recharge flow system in the aquifer.

Temporary peaks or rises in the groundwater levels of Figures 32 and 33 are correlated with flooding periods for nearby basins (basin 1 for North Well, basins 2 and 3 for Center Well and 18-, 24-, and 30-m Wells, and basin 4 for South Well). The rise in April 1980 of the water level in the 18-m Well was used to calculate the transmissivity of the aquifer from groundwater recharge theory (see Section 7, Aquifer Tests).

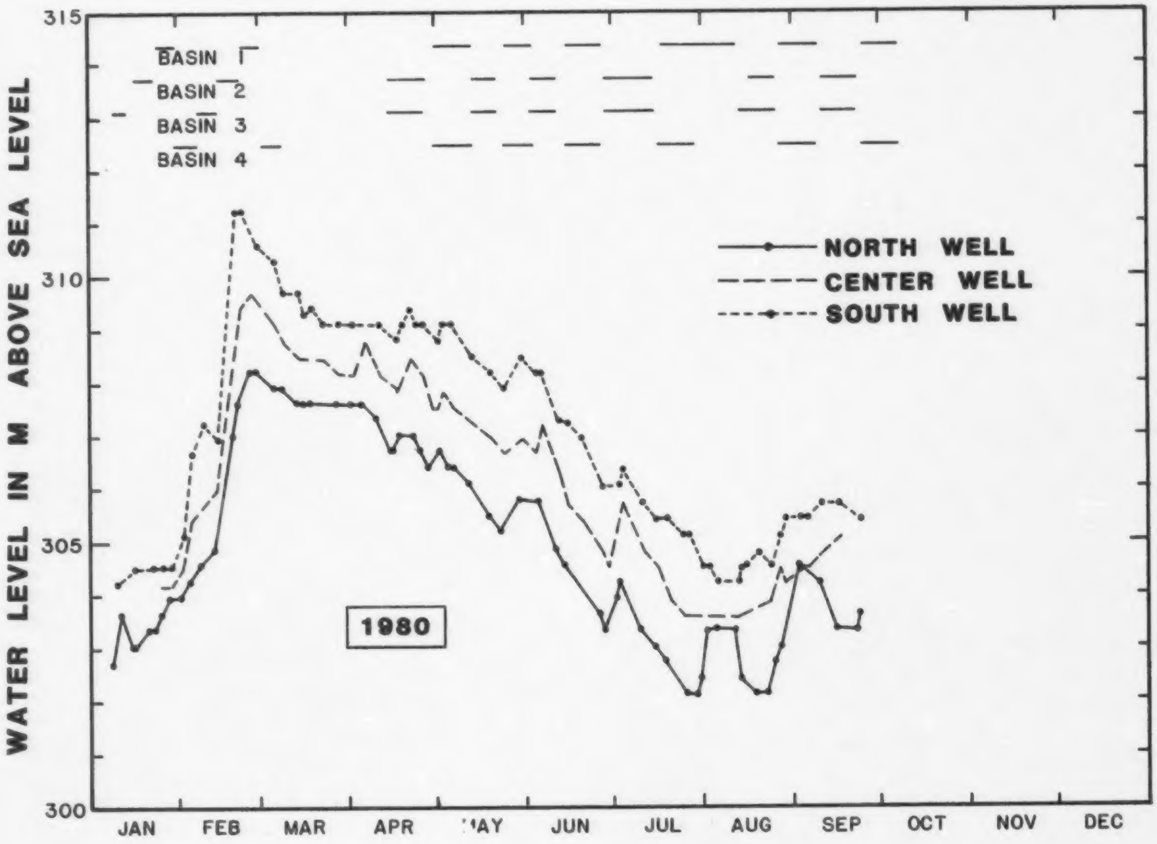

Figure 32. Water level elevations in the South, Center and North Wells in 1980.

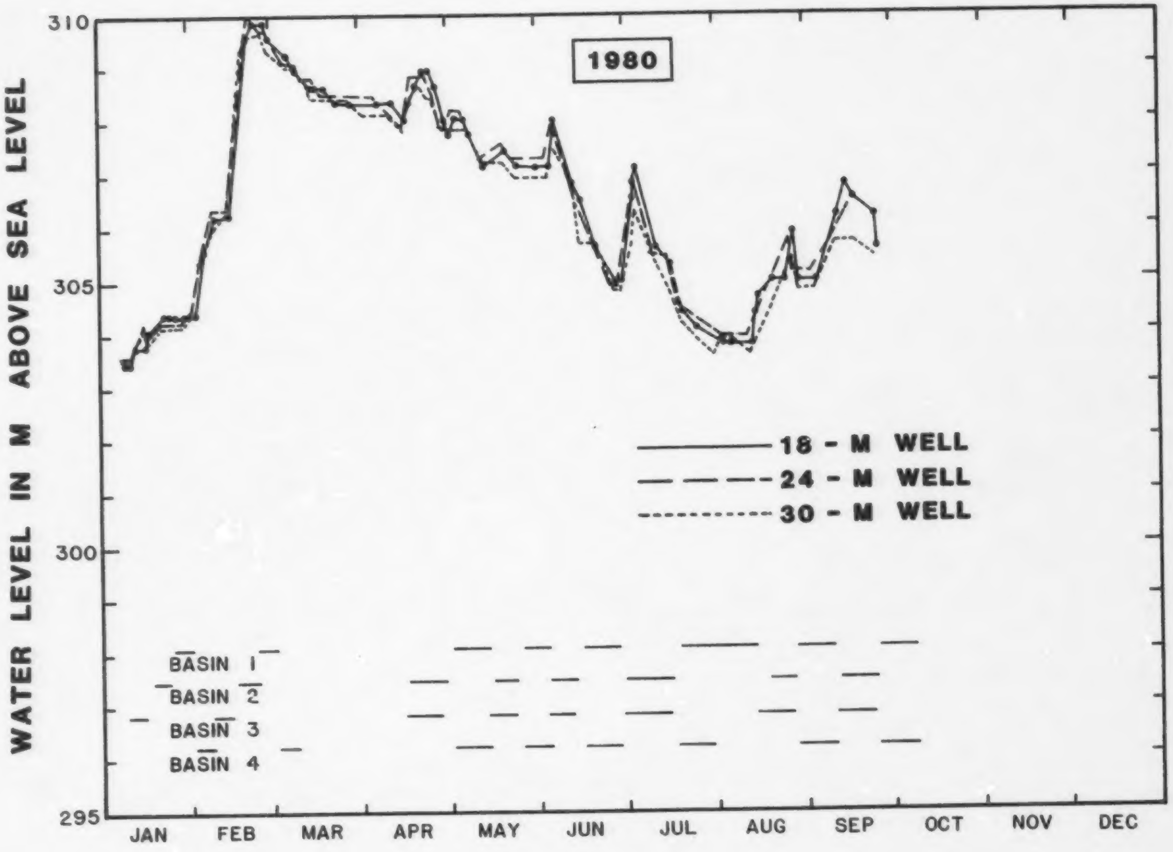

Figure 33. Water level elevations in the 18-, 24-, and 30-m Wells for 1980.

Selected profiles of groundwater levels across the infiltration basins from south to north are shown in Figure 34. The predominant groundwater movement again was from south to north, except for the profiles on 18 August, 15 September, and, to a lesser degree, 11 July. These three profiles all are associated with flooding of basins 2 and 3. Since these basins are in the center and the infiltration rates were high following the ripping of the surface soil in July, the water table in the center rose high enough to form a typical groundwater mound with lateral flow in both directions away from the center.

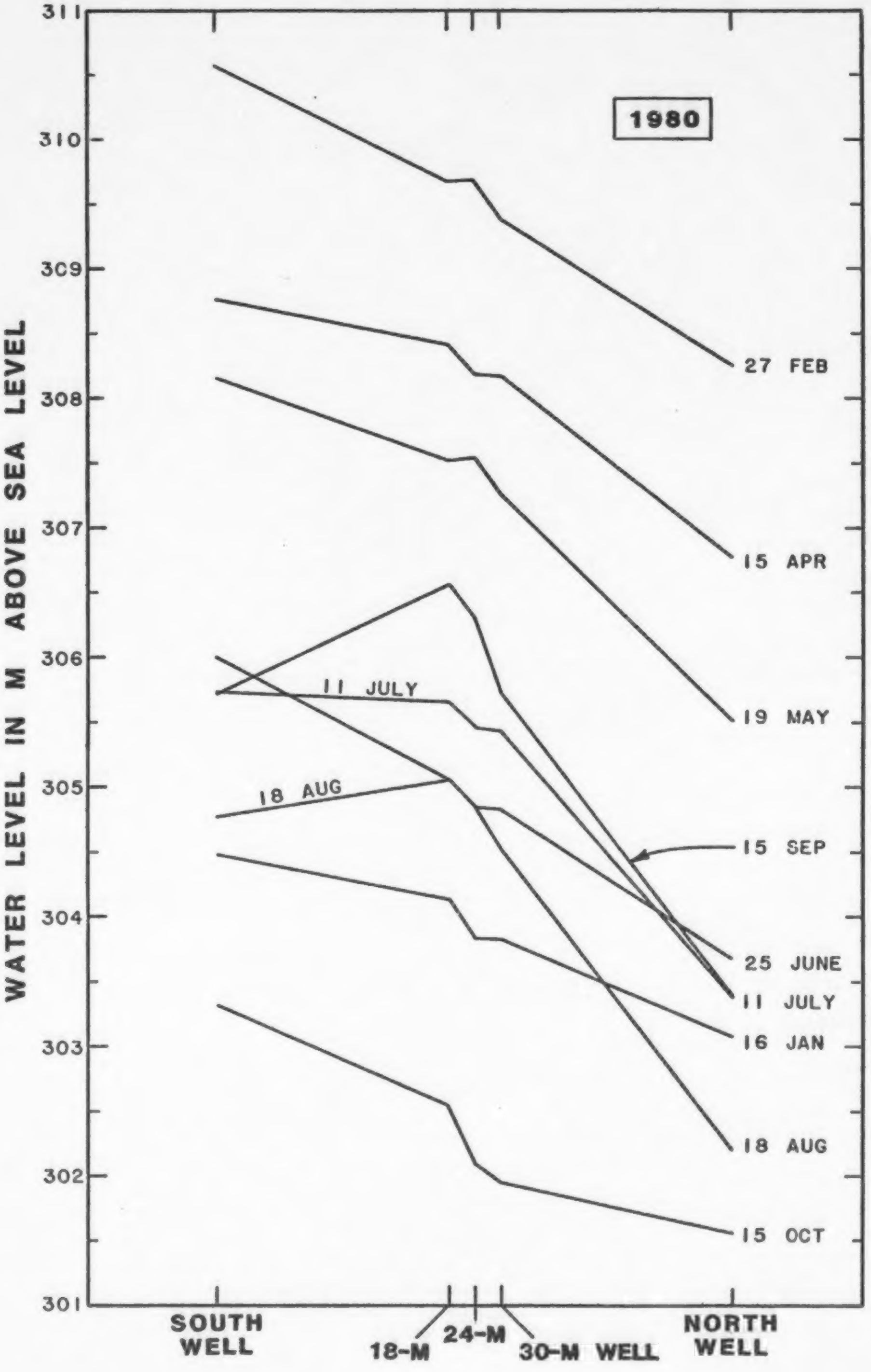

Figure 34. Groundwater level profiles from south to north midway across the basins in 1980.

SECTION 7

AQUIFER TESTS

Upon completion of the Center Well and prior to placing the pump, the hydraulic conductivity (K) of the aquifer around the perforated portion of the well was determined with a slug test on 30 July 1975 using the procedure of Bouwer and Rice (see Bouwer, 1978, and references therein). The equation for calculation of K with the slug method is:

$$K = \frac{r_c^2 \ln (R_e/r_w)}{2L_e} \frac{1}{t} \ln (y_o/y_t)$$

where r_c = radius of well section where water level rise is measured,
R_e = effective radial distance over which the head difference y is dissipated,
r_w = effective radius of well (well radius plus envelope or developed zone),
L_e = height of perforated, screened, or otherwise open section of well through which groundwater enters,
y = vertical distance of water level in well below static water level (y_o at t=0, y_t at t), and
t = time since y_o.

Values of R_e were determined by electric analog and can be evaluated from graphs in relation to geometry factors (Bouwer, 1978).

For the slug test, the water level in the well was lowered 7 cm by the rapid removal of a submerged cylinder. The subsequent rise was measured with a Statham Model 131 pressure transducer whose output was recorded on an Esterline Angus Model MS401BB millivolt recorder. The resulting time and water-level readings produced a straight line on semilog paper (Figure 35), in accordance with the theory. The test yielded a K-value of 29.2 m/day, which is between the horizontal and vertical K-values of 86 and 5.4 m/day, respectively, found at Flushing Meadows, and also within the range of the 10 to 236 m/day obtained with the individual well tests at Flushing Meadows (Bouwer et al., 1974). Since the Center Well had not yet been pumped, some drilling mud and other fines may still have been present in the aquifer around the well at the time of the slug test. Assuming an aquifer thickness of 500 m, the transmissivity of the aquifer below the 23rd Avenue basins would be 14,600 m^2/day.

After the pump and power unit was installed (see Figure 36 for pertinent dimensions), a step-drawdown test was performed on the Center Well

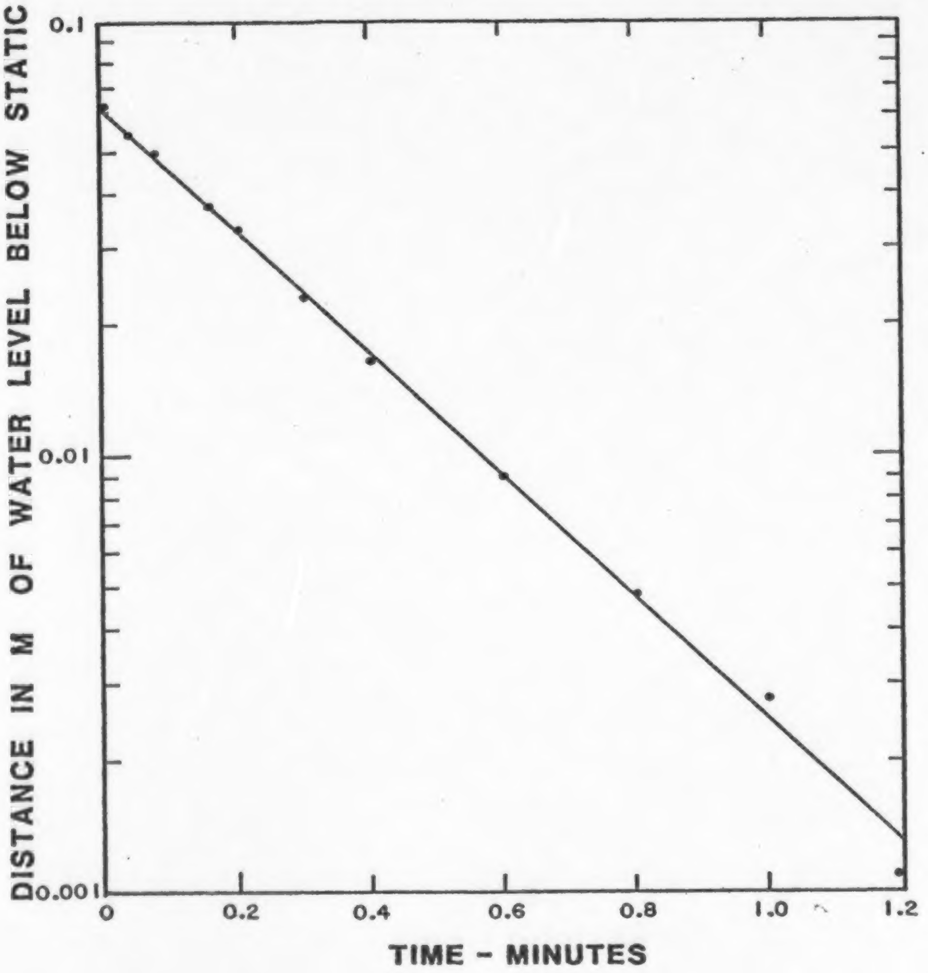

Figure 35. Water level rise in Center Well for slug test.

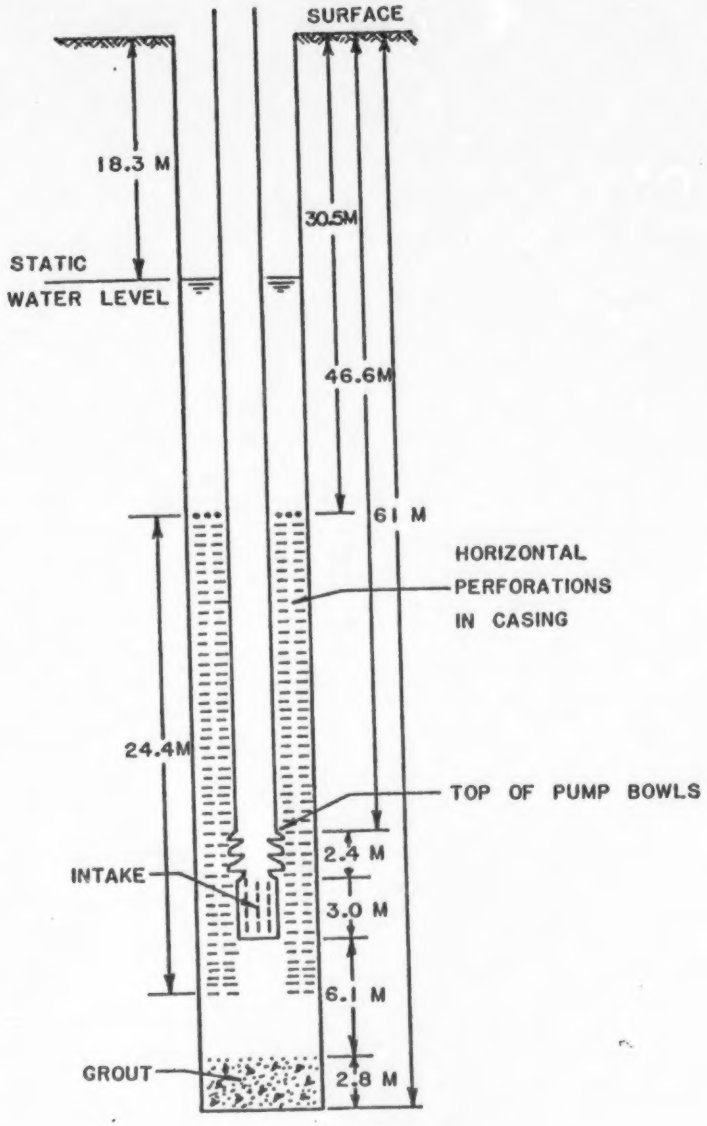
SURFACE
18.3 M
30.5M
STATIC WATER LEVEL
46.6M
61 M
HORIZONTAL PERFORATIONS IN CASING
24.4M
TOP OF PUMP BOWLS
2.4 M
INTAKE
3.0 M
6.1 M
GROUT
2.8 M

Figure 36. Perforated section, pump placement, and other pertinent dimensions for the Center Well.

on 12 November 1975. The well pumping rate was increased about every 20 minutes while measuring the depth of the water level inside the well with a bubble tube. The resulting step-drawdown curve (Figure 37) was analyzed with the logarithmic trial and error technique of Rorabough (see Bouwer, 1978, and references therein) to determine the coefficients and exponents in the equation

$$s_{iw} = C_f Q + C_w Q^n,$$

where s_{iw} = total drawdown inside well,
C_f = formation factor,
C_w = well-loss factor,
n = exponent for turbulent flow ($n > 2$), and
Q = discharge.

The analysis yielded the following values:

$C_f = 4.34 \times 10^{-4}$ day/m^2,
$C_w = 1.37 \times 10^{-19}$ day/m^2, and
$n = 4.79$.

Using these values, the total drawdown was broken down into the head loss (s_f) in the formation and the head loss (s_w) due to entry into the well (Figure 38). The sum of s_f and s_w (dashed line) agreed with the observed total drawdowns. The curves show that well losses become quite significant when $Q > 10{,}000$ m^3/day.

Since $C_f = \ln(r_2/r_w)/2\pi T$ (according to Thiem equation), where r_w is the effective well radius and r_2 is the radius of influence of the well, the transmissivity (T) can be calculated from C_f if r_2 is known. Assuming $r_2 = 100$ m and $r_w = 0.5$ m, this yields an effective T of 1900 m^2/day. Since the actual aquifer is much thicker than the perforated height of the well, T of the entire aquifer is much higher. Again assuming that the aquifer is 500 m in thickness, the actual transmissivity would then be more on the order of 30,000 m^2/day (using the theory of partially penetrating wells by Sternberg; see Bouwer, 1978, and references therein). As can be expected, this value is somewhat less than the value of 39,000 m^2/day that would be obtained by multiplying T from the step-drawdown test directly by the ratio of aquifer thickness to height of perforated section of well (i.e., $1900 \times 500/24.4 = 38{,}934$).

A Theis pumping test was performed on the Center Well on 5 June 1979, using the North Well as observation well for the water table drawdowns. The data were analyzed with the Chow method (Bouwer, 1978, and references therein), which is a graphical procedure that eliminates the need for curve matching. A plot of drawdown (s) against the logarithm of pumping time (t) is shown in Figure 39. Taking the tangent to the curve at $s = 0.07$ m, the slope Δs_A of the tangent was 0.098 m per log cycle, which gave a value of $0.07/0.098 = 0.71$ for F(u) in Chow's graph (Figure 5.4 in Bouwer, 1978). The resulting value of u was 0.16 and that of W(u) was 1.5, which when substituted with $Q = 10{,}000$ m^3/day,

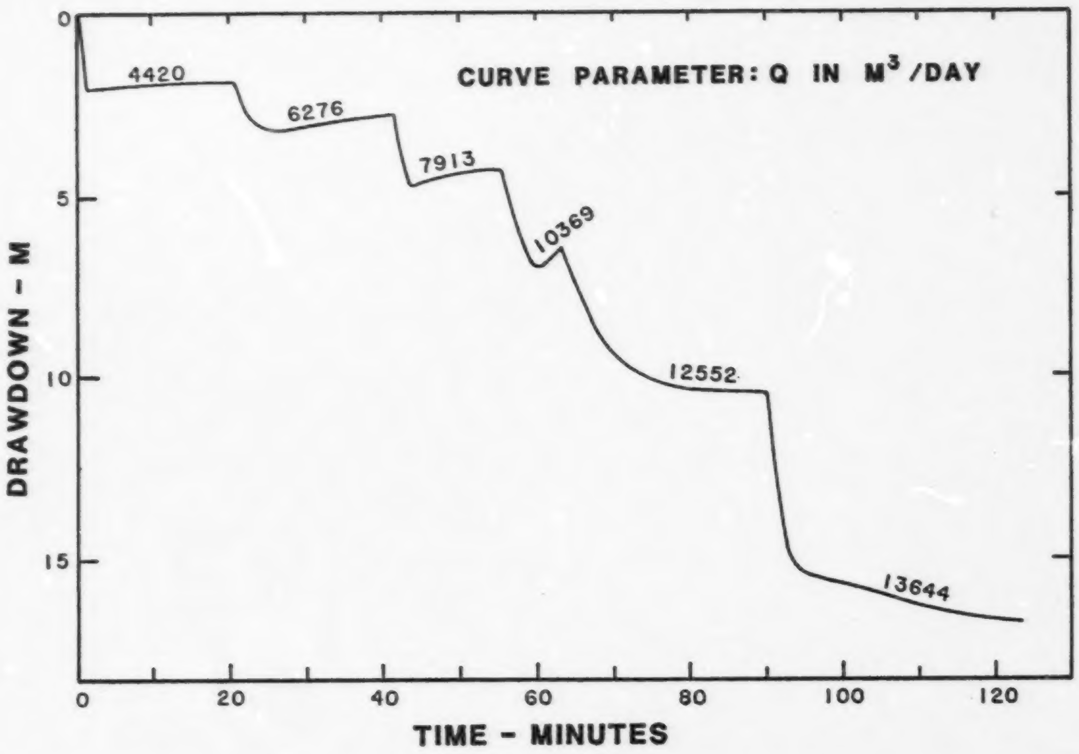

Figure 37. Water level drawdown in Center Well at different pumping rates for step-drawdown test.

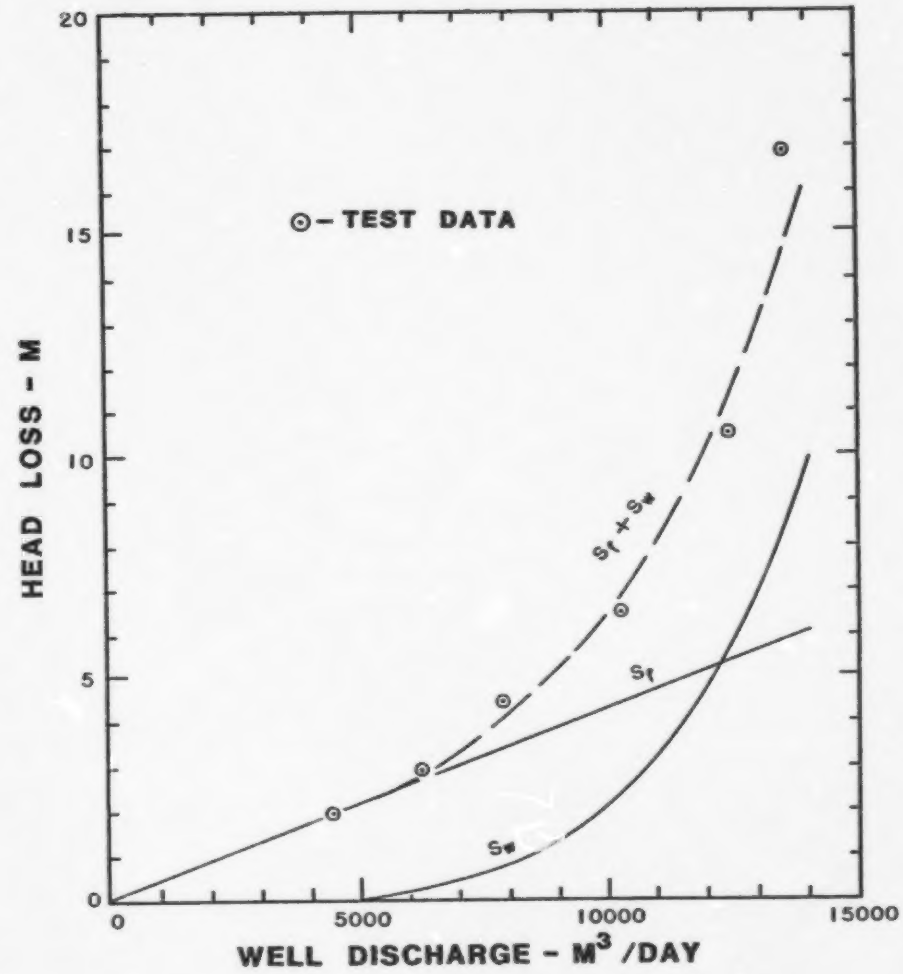

Figure 38. Total drawdown (dashed line) in Center Well as sum of head loss in aquifer (s_f-curve) and entry losses (s_w-curve) in relation to well discharge.

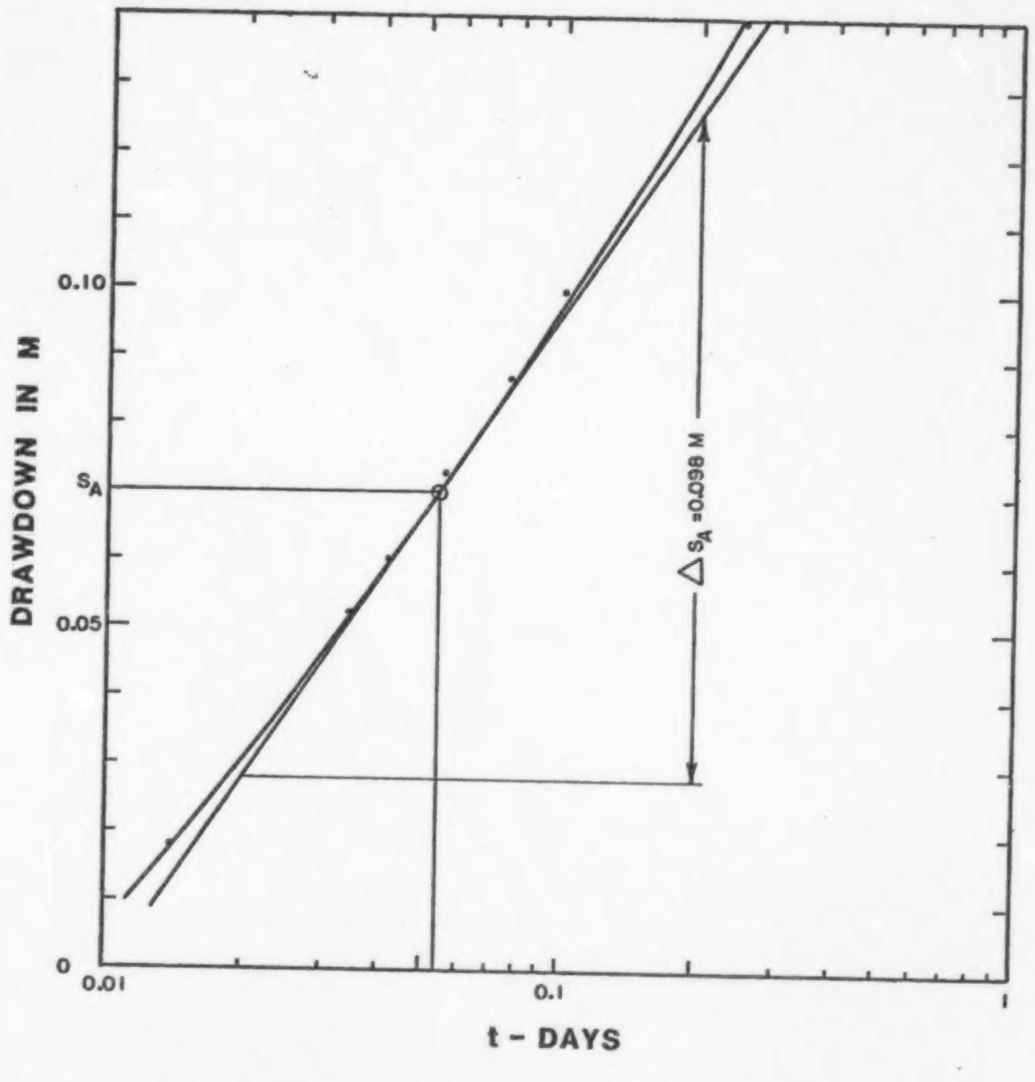

Figure 39. Drawdown in North Well versus log t for Center Well pumping test.

$r = 200$ m, $s = 0.07$ m, and $t = 0.054$ days in the equations for transmissivity T and specific yield S gave $T = 17{,}000$ m^2/day, and $S = 0.015$. Again this T-value must be corrected for the partial penetration of the Center Well to get an estimate of T of the entire aquifer with an assumed thickness of 500 m. Because T is based on drawdown data from observation wells, the flow system involved in the Theis pumping test is larger than that in the step-drawdown test. Thus, the correction factor probably is less than that for the step-drawdown method. However, the resulting T for the entire aquifer could well be on the order of 100,000 m^2/day. The specific yield of 0.015 is very low for an unconfined aquifer. However, this value may not be all that unreasonable in view of the preponderance of gravel and boulders in the aquifer with sand and silt filling up the voids (Figure 2). Also, the deeper aquifer may be confined or semiconfined in some places.

A fourth estimate of aquifer transmissivity was obtained from the rise of the groundwater mound below the infiltration basins, using the method of Bianchi and Muckel (Bouwer, 1978, and references therein). Figure 33 shows that the water level in the 18-m Well rose 1 m in 10 days in the second half of April 1980, due to flooding of basins 2 and 3. The average infiltration rate for the two basins during that period was 0.33 m/day (Figures 23 and 24). To calculate T of the aquifer, the fillable porosity in the zone of water table rise must be known. Because of hysteresis, the fillable porosity tends to be less than the specific yield. The specific yield of 0.015 obtained with the Theis pumping test, however, may be on the low side. Arrival times of NO_3-N peaks in the North Well indicated that volumetric water contents were on the order of 7% in the vadose zone and 14% in the aquifer, which would give a fillable porosity of 14-7 = 7% (see section Nitrogen in Effluent and Renovated Water). Based on these data, a fillable porosity of 5% was assumed in the calculation of aquifer transmissivity from mound rise.

Bianchi and Muchel presented a dimensionless graph (Fig. 8.21 in Bouwer, 1978) relating $h_t\ f/vt$ to $W/(4\ Tt/f)^{1/2}$ for different values of L/W, where

h_t = rise of mound center above static water table at time t,
f = fillable porosity,
v = infiltration rate,
t = time since infiltration began,
W = width of flooded area,
L = length of flooded area, and
T = transmissivity of aquifer.

Since for the case at hand h_t=1 m, f=0.05, v=0.33 m/day, and t=10 days, the parameter $h_t\ f/vt=0.015$. Bianchi and Muchel's graph shows that for L/W=465/188=2.5, the corresponding value of $W/(4Tt/4)^{1/2}=0.023$ (assuming linearity of the curves near the origin). Substituting the values of W, T, and f in this parameter then yields T=86,400 m^2/day. This value is between the T-values from the slug test and the Theis pumping test.

The difference between the T-values from the various tests is undoubtedly due to assumptions and methodology, particularly as they affect the portion of the aquifer involved in the flow system and the direction and distribution of the flow therein. For purposes of design and prediction of groundwater mound behavior in the area, a T-value of 50,000 m^2/day and an f or S-value of 0.05 may be reasonable and probably will be on the conservative side.

SECTION 8

WATER QUALITY STUDIES

METHODS

Suspended solids were determined by filtering through glass fiber filters, drying, and weighing.

Total dissolved solids (TDS) were determined by conductivity reading and multiplying millimho's by 640.

pH was read directly with the Corning pH meter.

Nitrate and nitrite were determined with the Technicon AutoAnalyzer I. In the procedure the nitrate is reduced to nitrite by an alkaline solution of hydrazine sulfate containing a copper catalyst followed by treatment with sulfanilamide under acidic conditions forming a diazo compound which coupled with N-1 naphthelethylenediamine dihydrochloride forms a soluble dye.

Ammonium was determined with the Technicon AutoAnalyzer I procedure which utilizes the Berthelot reaction. An indophenol related compound forms from the addition of an ammonium salt to sodium phenoxide followed by the addition of sodium hypochlorite.

Organic-N was determined with the Technicon AutoAnalyzer I procedure using the continuous digester. The amount of ammonia produced was then measured.

Orthophosphate was determined with the Technicon AutoAnalyzer I procedure whereby ammonium molybdate reacts in acid medium to form molybdophosphoric acid which is subsequently reduced to molybdenum blue complex by reaction with amino-naphtholsulfonic acid reagent.

Fluoride was determined by the SPADNS method. A red color is formed with SPADNS reagent and zirconium acid. The color is diminished by fluoride.

Boron was determined manually by the curcumin colorimetric method. The sample was acidified and evaporated in the presence of curcumin forming rosocyanine (a red colored product).

Fecal coliform counts were determined by the millipore method using M-FC broth and 24-hour incubation at 44.5° C.

Total organic carbon was determined with the Technicon AutoAnalyzer II. The inorganic carbon is removed by purging with sulfuric acid. The sample is then mixed with sulfuric acid and potassium persulfate and subjected to ultraviolet light. The carbon dioxide generated from the organic carbon is passed through a dialysis membrane. The resultant product uses phenolphthalien indicator to identify the quantity of carbon.

TYPICAL QUALITY OF SECONDARY SEWAGE EFFLUENT IN PHOENIX

An example of the chemical and biological composition of Phoenix's secondary sewage effluent (activated sludge, unchlorinated) is given in Table 4. Most of the data were obtained for the 91st Avenue Treatment Plant. More detailed discussions of the various constituents and their fate in the soil and aquifer system are presented in the rest of this section.

SUSPENDED SOLIDS

Suspended solids contents of the secondary effluent entering and leaving the 32-ha lagoon are shown in Figure 12, which applies to 1975 and 1976 when the effluent was passed through the lagoon before it entered the infiltration basins. Suspended solids contents of the secondary effluent entering the 32-ha pond were higher in the winter than in the summer. In the winter, the 32-ha pond functioned as a sedimentation basin, as indicated by the lower suspended solids content in the outflow from the pond (which was the inflow for the infiltration basins). In the summer, however, the suspended solids content of the outflow from the 32-ha pond was considerably higher than that of the inflow. As discussed in Section 5, the increase in suspended solids content in the summer was due to algae growth in the 32-ha pond. The detention time in this pond was on the order of three days, enough for the algae to grow and reach bloom proportions. Additional algae growth took place in the infiltration basins themselves, where water turnover times again were on the order of days due to the high water depth (0.9 m) and low infiltration rates (about 0.15 m/day). Thus, while the suspended solids content of the effluent water in the infiltration basins was around 25 mg/ℓ in the winter, it was at least 50 to 100 mg/ℓ in the summer and probably much higher than that if additional growth of algae in the infiltration basins is taken into account.

After the construction of the bypass channel around the 32-ha pond and conveying the secondary effluent directly from the inlet channel to the infiltration basins, the suspended solids content of the effluent entering the infiltration basins was much lower (Figures 40, 41, 42, and 43), and averaged 9.6 mg/ℓ for the second half of 1979. The suspended solids content of the secondary effluent leaving the basins at the outflow ends (Figure 1) generally was slightly lower than that of the effluent entering the basins (Figures 40, 41, 42, and 43). This was due to further settling of suspended solids in the infiltration basins. Occasional increases in suspended solids contents were also observed as

TABLE 4. CHEMICAL AND BIOLOGICAL COMPOSITION OF SECONDARY SEWAGE EFFLUENT FROM PHOENIX TREATMENT PLANTS (GENERAL RANGES OR AVERAGES IN MG/ℓ OR UNITS AS INDICATED)

Total dissolved solids	1,000
Calcium	82
Magnesium	36
Sodium	200
Potassium	8
Bicarbonate	381
Chloride	213
Sulfate	107
Phosphate-phosphorus	9
Ammonium-nitrogen	10-35
Nitrate-nitrogen	3
Nitrite-nitrogen	trace
Organic nitrogen	4
Boron	0.6
Fluoride	2
Cadmium	0.008
Copper	0.123
Lead	0.082
Mercury	0.002
Zinc	0.193
pH (units)	8
BOD_5	10-20
COD (dichromate)	30-60
Total organic carbon	19
Fecal coliform bacteria (per 100 mℓ)	10^5-10^6
Viruses (PFU's per liter)	21

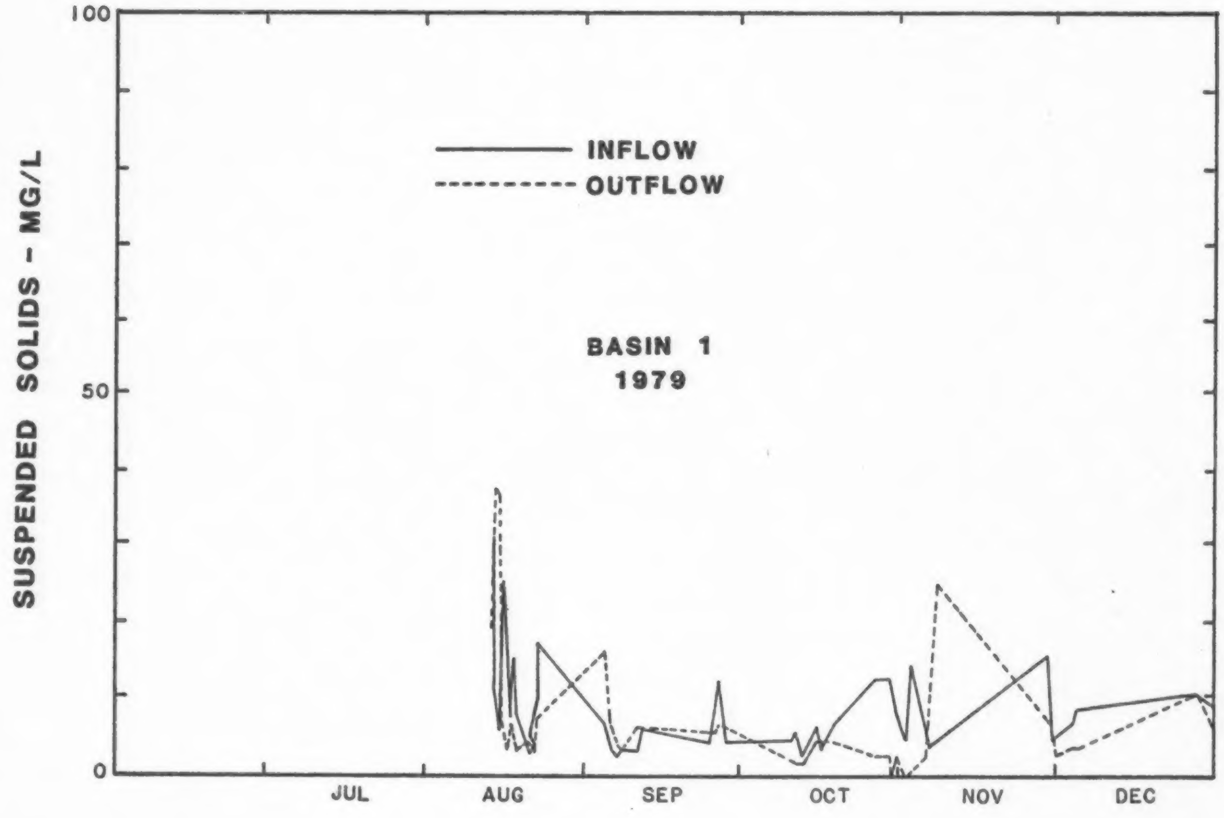

Figure 40. Suspended solids content of secondary effluent in 1979 at inflow and outflow ends of basin 1.

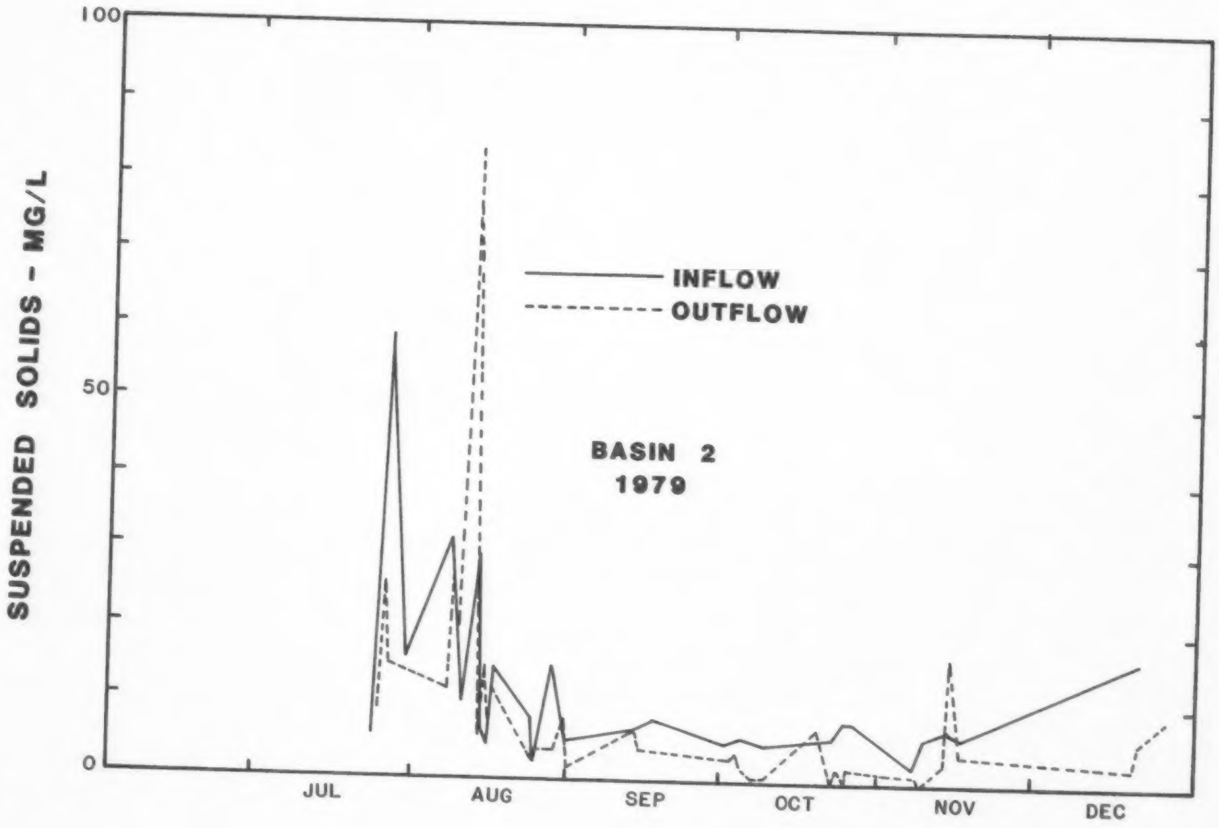

Figure 41. Suspended solids content of secondary effluent in 1979 at inflow and outflow ends of basin 2.

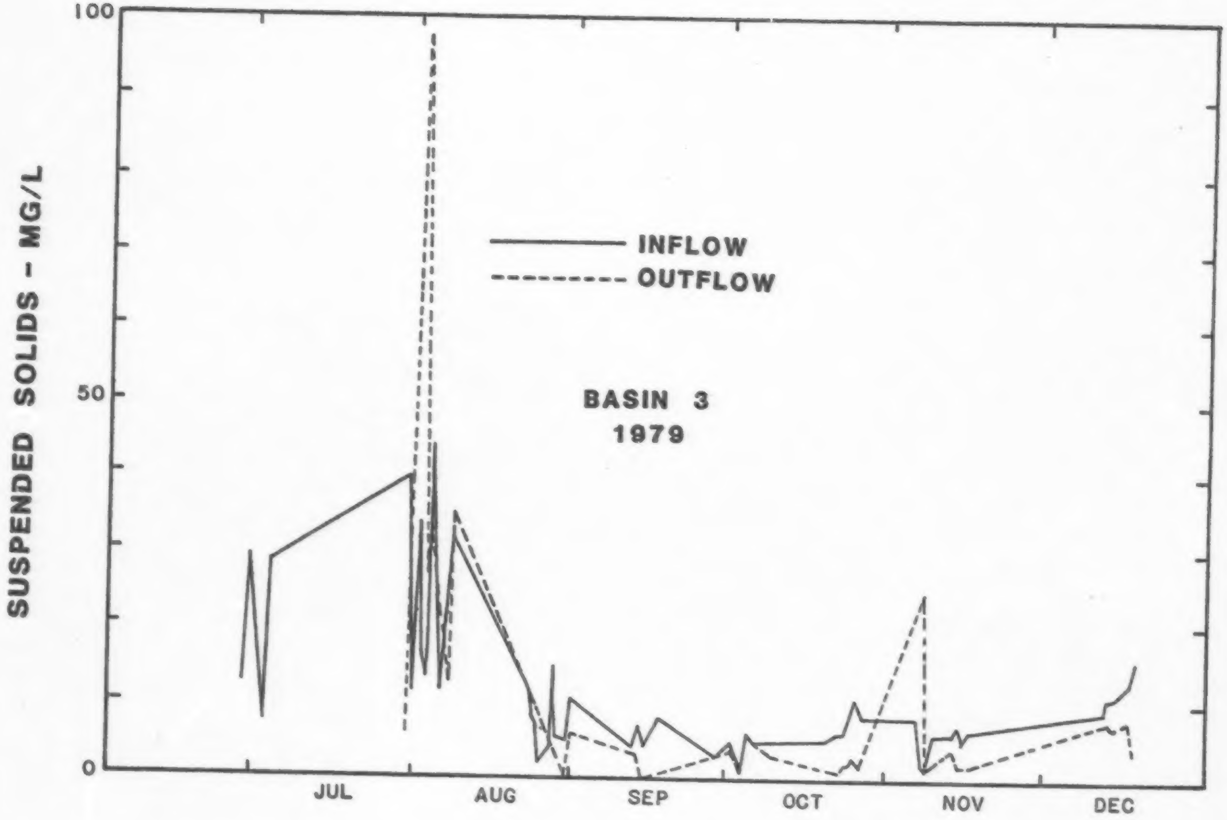

Figure 42. Suspended solids content of secondary effluent in 1979 at inflow and outflow ends of basin 3.

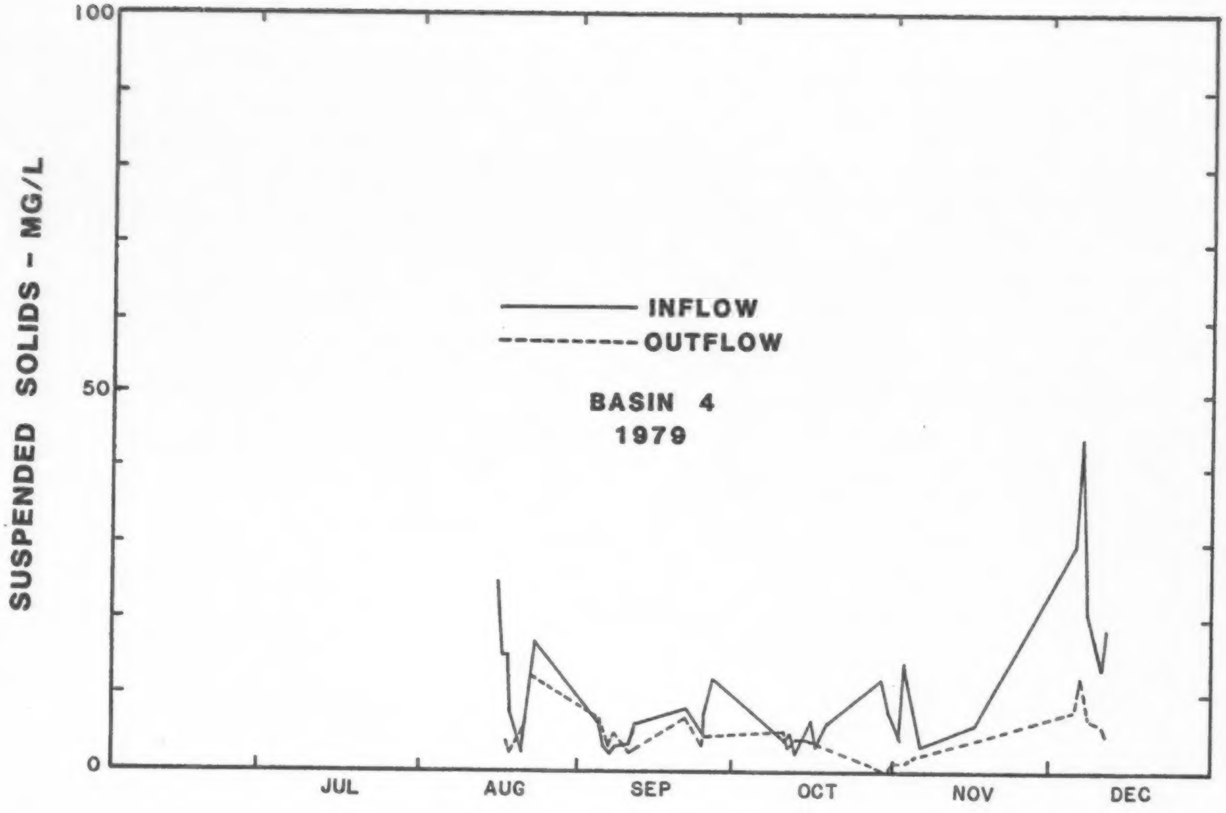

Figure 43. Suspended solids content of secondary effluent in 1979 at inflow and outflow ends of basin 4.

the secondary effluent passed through the infiltration basins (see, for example, the graphs for basins 1, 2, and 3). This was due to growth of algae in the deeper and more stagnant areas of the basins near the outflow structures. Suspended solids contents of the renovated water in 1979 averaged 4.3 mg/ℓ for the North Well and 0.5 mg/ℓ for the Center Well. The higher value for the North Well was probably due to the fact that this well had a lower specific capacity (more drawdown) than the other observation wells, indicating the presence of finer soil materials below the well bottom that could contribute to a higher sediment content of the water pumped from that well.

In 1980, the suspended solids content of the sewage effluet entering the infiltration basins generally was in the range of 5 to 25 mg/ℓ, and averaged 14.2 mg/ℓ (Figure 44). Thus, the bypass channel continued to be effective in minimizing the algae growth and suspended solids content of the effluent entering the infiltration basins. The suspended solids content of the renovated water averaged 1.3 mg/ℓ for the Center Well. Individual concentrations did not exceed a value of 5 mg/ℓ (Figure 44). The same was true for the 18-m Well (Figure 45), where the average suspended solids content in 1980 was 2.0 mg/ℓ.

DISSOLVED SOLIDS

The TDS contents of the sewage effluent and of the renovated water were essentially the same. In 1976, they averaged 910 mg/ℓ. In 1979, there was greater fluctuation and TDS contents generally ranged between 650 and 900 mg/ℓ for effluent and renovated water (Figure 46). The average for the effluent was 756 mg/ℓ. In 1980, the TDS content of the sewage effluent ranged between 600 and 800 mg/ℓ (Figure 47), and averaged 698 mg/ℓ. Average TDS concentrations of the renovated water in 1980 were 738 mg/ℓ for the Center Well, 769 mg/ℓ for the 18-m Well, and 778 mg/ℓ for the North Well. At a loading rate of 100 m/year and an evaporation of 1.8 m/year (based on annual lake evaporation) from the soil and water surface in the infiltration basins, evaporation would account for about 15 mg/ℓ of the roughly 60 mg/ℓ average difference between the TDS content of sewage effluent and renovated water. The rest of the increase may be due to mobilization of calcium carbonate in the soil and aquifer as the pH of the effluent dropped from slightly alkaline (about 8) to neutral. Variations in TDS contents and different times of sampling may also have played a role.

The TDS concentrations of the effluent and renovated water are in the low part of the 500 to 2000 mg/ℓ range where increasing problems of salinity can be encountered when the water is used for irrigation (Ayers, 1975; see also Bouwer, 1978). Thus, with proper irrigation management, the renovated water could be used to irrigate salt sensitive crops.

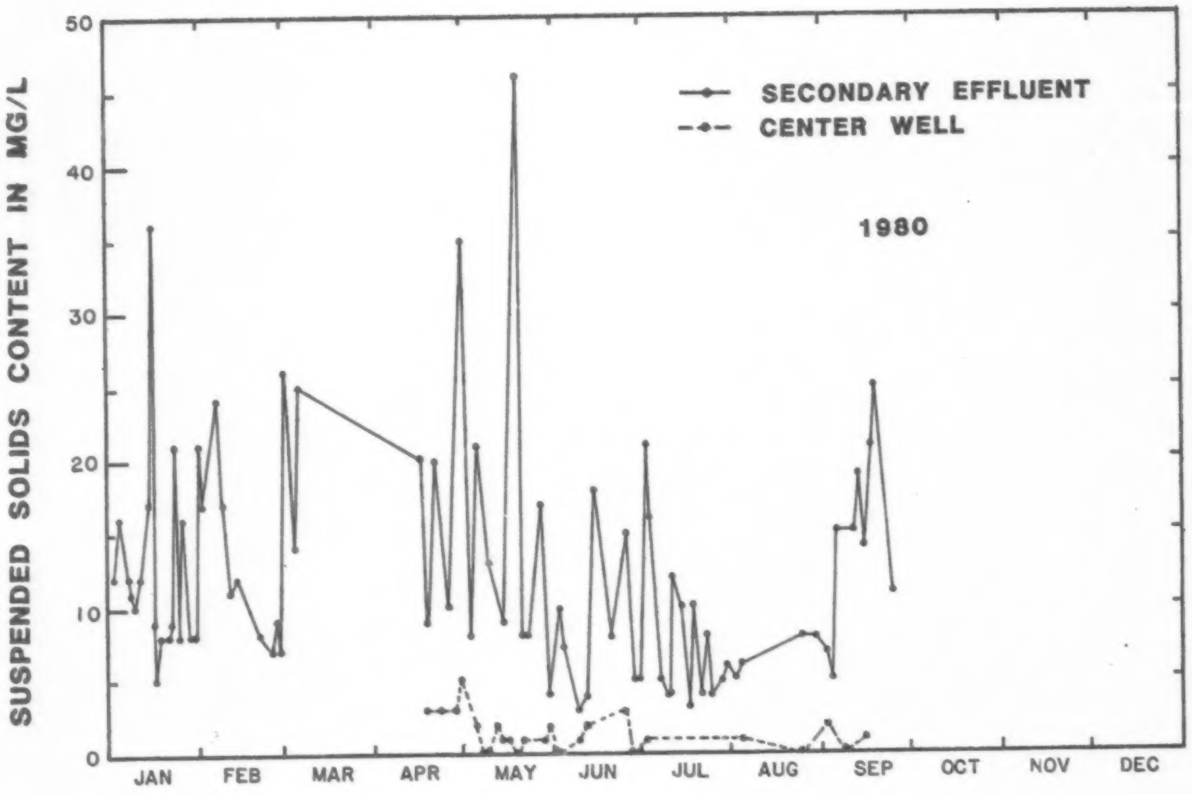

Figure 44. Suspended solids content of secondary effluent entering basins and of renovated water from the Center Well in 1980.

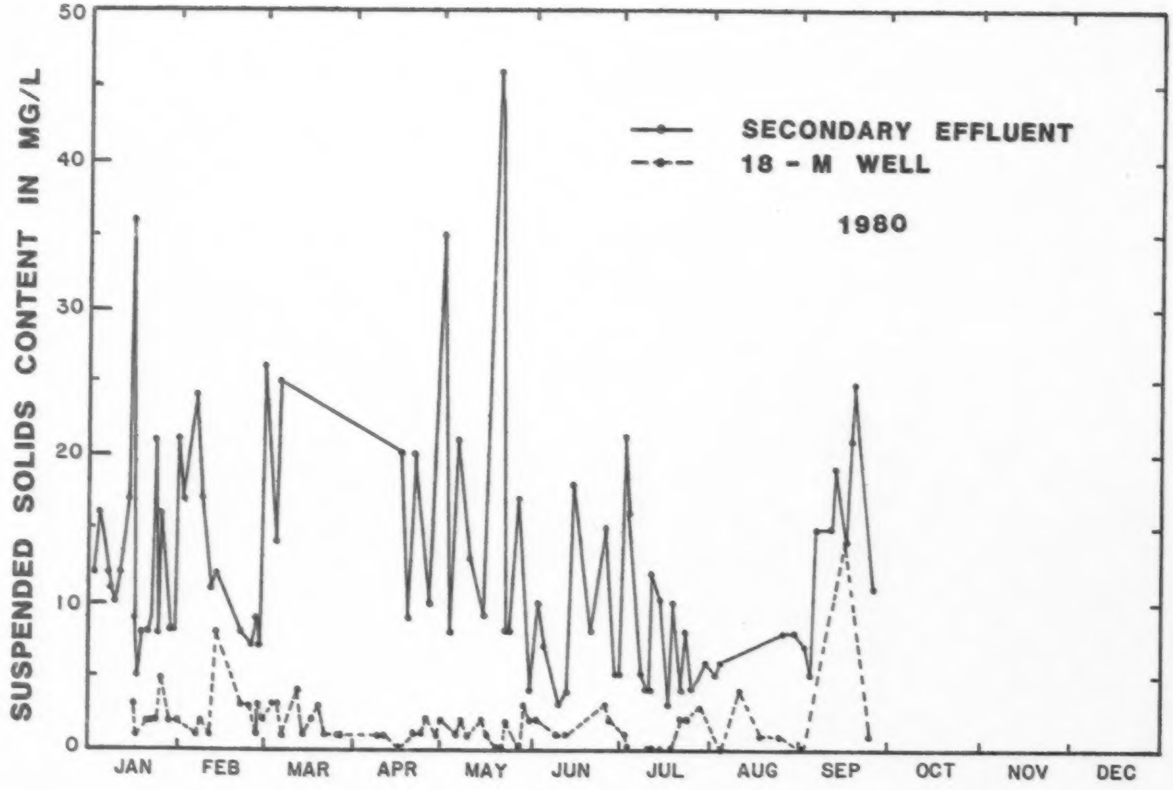

Figure 45. Suspended solids content of secondary effluent entering basins and of renovated water from the 18-m Well in 1980.

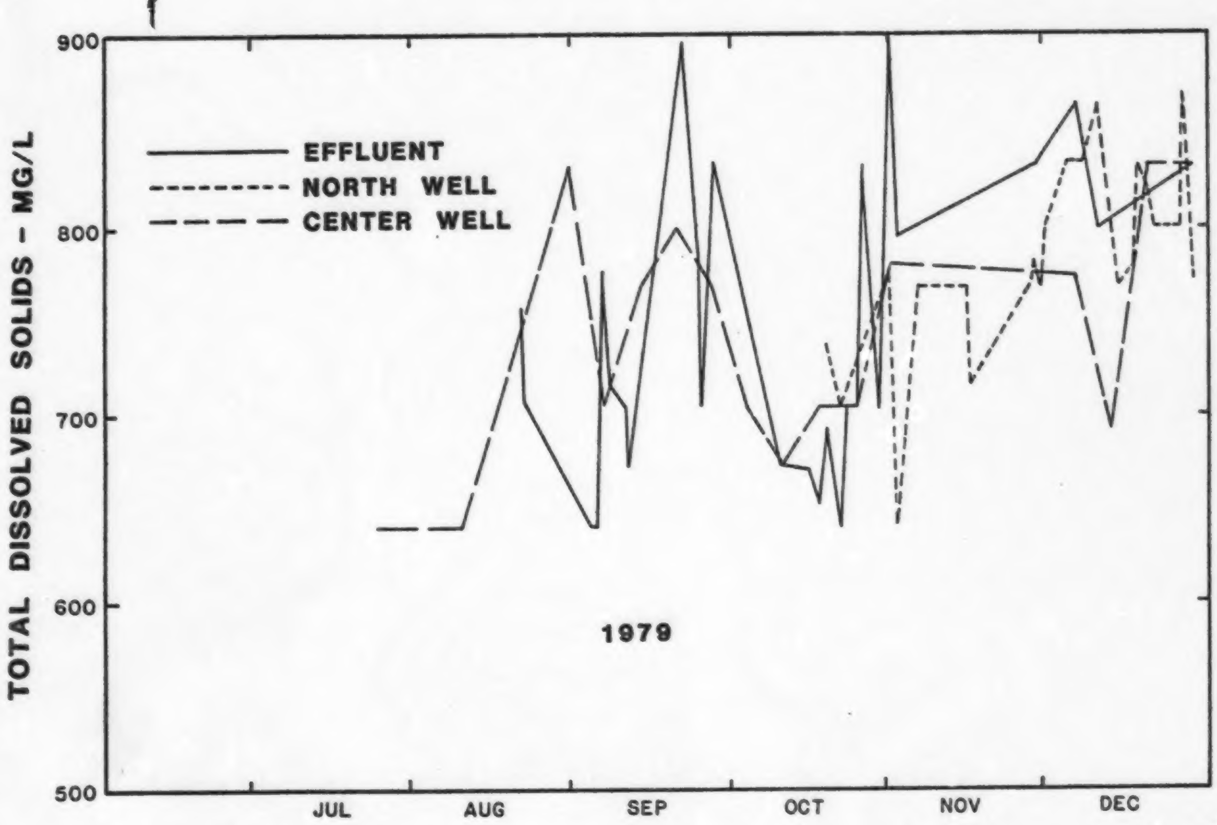

Figure 46. TDS concentrations in secondary effluent and in renovated water from North Well and Center Well, 1979.

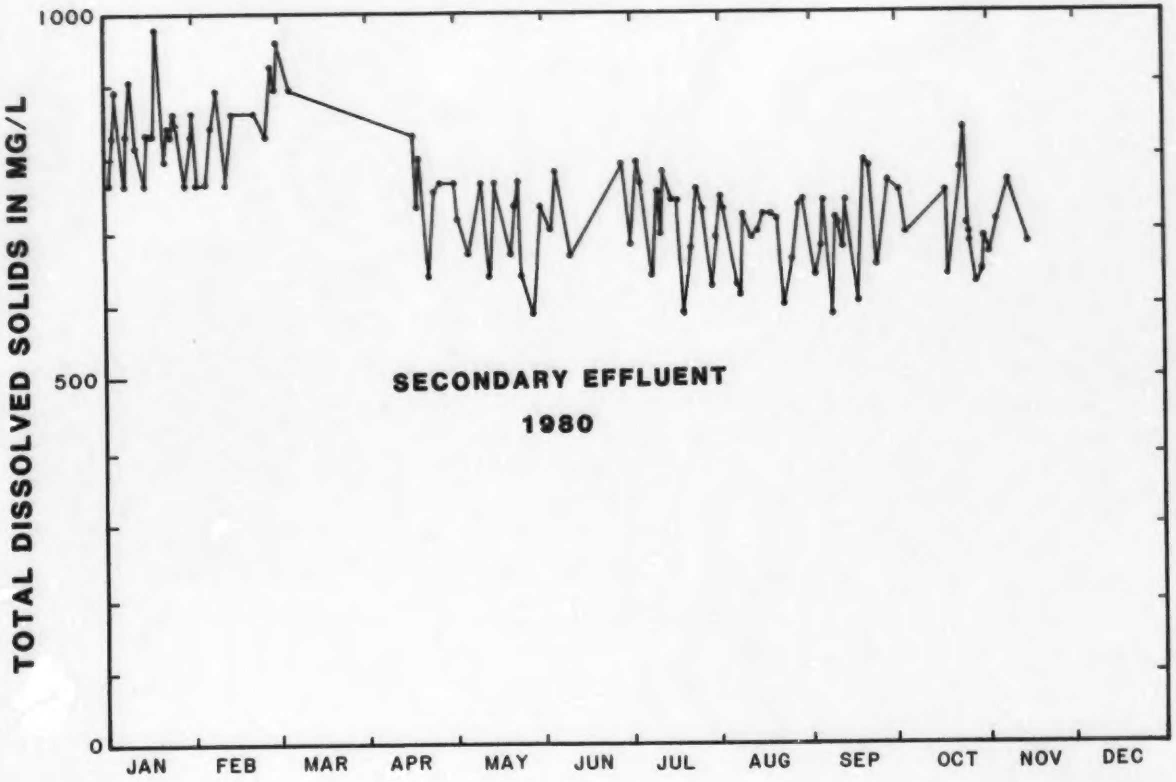

Figure 47. TDS concentrations in secondary effluent, 1980.

pH VALUES

As at the Flushing Meadows Project, the secondary effluent was slightly alkaline (pH about 8) and the renovated water essentially neutral (pH about 7). Algae growth in the 32-ha pond caused pH values of the effluent to increase to 9 and sometimes even higher as carbon dioxide was taken up for photosynthesis during the day. After the bypass channel was constructed and used, however, algae growth in the effluent was much reduced and the pH of the effluent in the infiltration basins remained around 8.

NITROGEN

Nitrogen in Effluent and Renovated Water

In the pre-bypass-channel-period, the total-N content of the secondary effluent entering the infiltration basins generally ranged between 15 and 35 mg/ℓ (Figure 48) and averaged about 24 mg/ℓ. The NO_3-N content of the water from the Center Well initially was relatively high (about 30 mg/ℓ), but decreased rapidly and then more gradually to a range of 4 to 6 mg/ℓ (Figure 48). This decrease was probably due to displacement of old nitrified seepage water in the aquifer from the original aerobic oxidation pond by renovated water after the pond was converted into a rapid-infiltration system. The NH_4-N content of the renovated water was below 1 mg/ℓ. The curves in Figure 48 indicate that the average total N content of the renovated water after February 1976 was about 75% lower than that in the effluent (i.e. about 5 mg/ℓ versus about 20 mg/ℓ).

In the post-bypass-channel period starting in June 1979, the total N content in the secondary effluent for the rest of 1979 generally ranged between 13 and 26 mg/ℓ (Figure 49) and averaged about 18 mg/ℓ, most of it in the ammonium form. Renovated water from the North Well showed the characteristic NO_3-N peaks when water that infiltrated at the beginning of a flooding period reached the intake of the well (Figure 50). This water generally had a high NO_3-N content because it leached out nitrate from the top m of the soil that was formed there during drying. The center of the NO_3-N peak in the beginning of November 1979 occurred seven days after the start of the preceding flooding period, which was on 26 October. The center of the NO_3-N peak in the first half of December occurred about 12 days after the start of the next flooding period, which was 27 November. Since the average infiltration rate for the 26 October flooding period was 0.3 m/day and for the 27 November flooding period 0.18 m/day, it should take the effluent water 0.3/0.18 times as long to travel from basin 1 to the North Well for the 27 November period as for the 26 October period. According to this, the arrival time of the NO_3-N peak at the North Well for the 27 November flooding period should have been 1/0.6 x 7 = 11.7 days after flooding began. This agrees with the observed period of 12 days. If the infiltration rate during the flooding period that started 27 November had also been 0.3 m/day, the center of the NO_3-N peak would have arrived at the North Well seven days after the start of flooding and the time period between the centers of

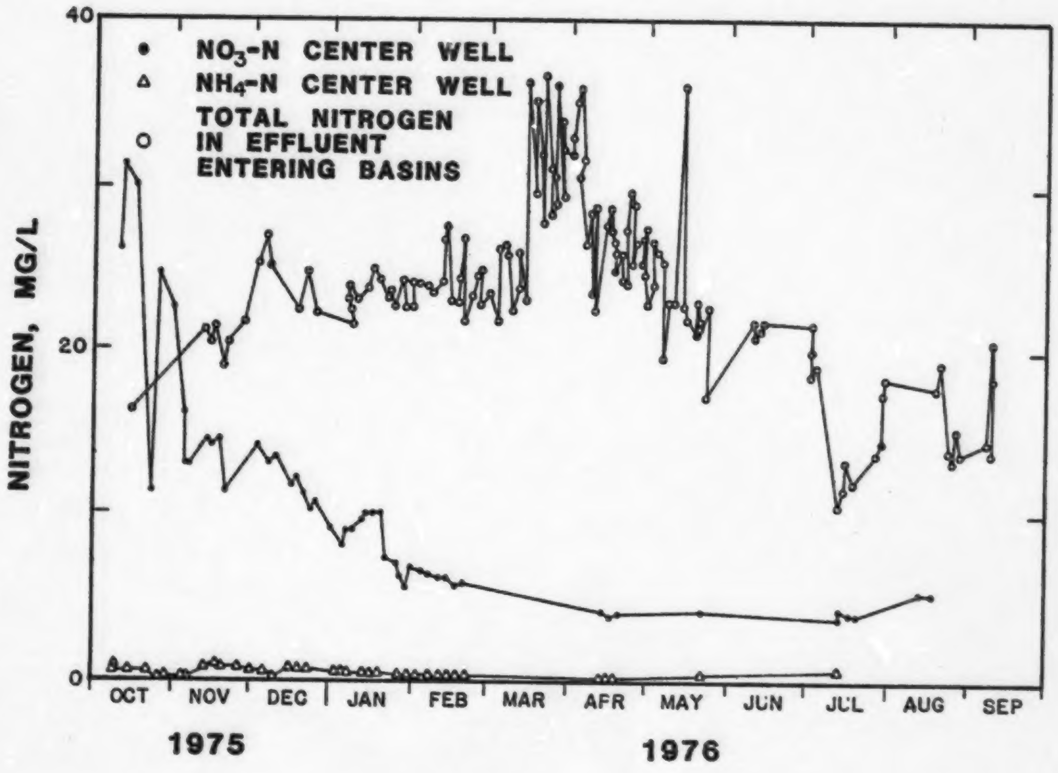

Figure 48. Total N in effluent entering infiltration basins and NO_3-N and NH_4-N in renovated water from Center Well, 1975-1976.

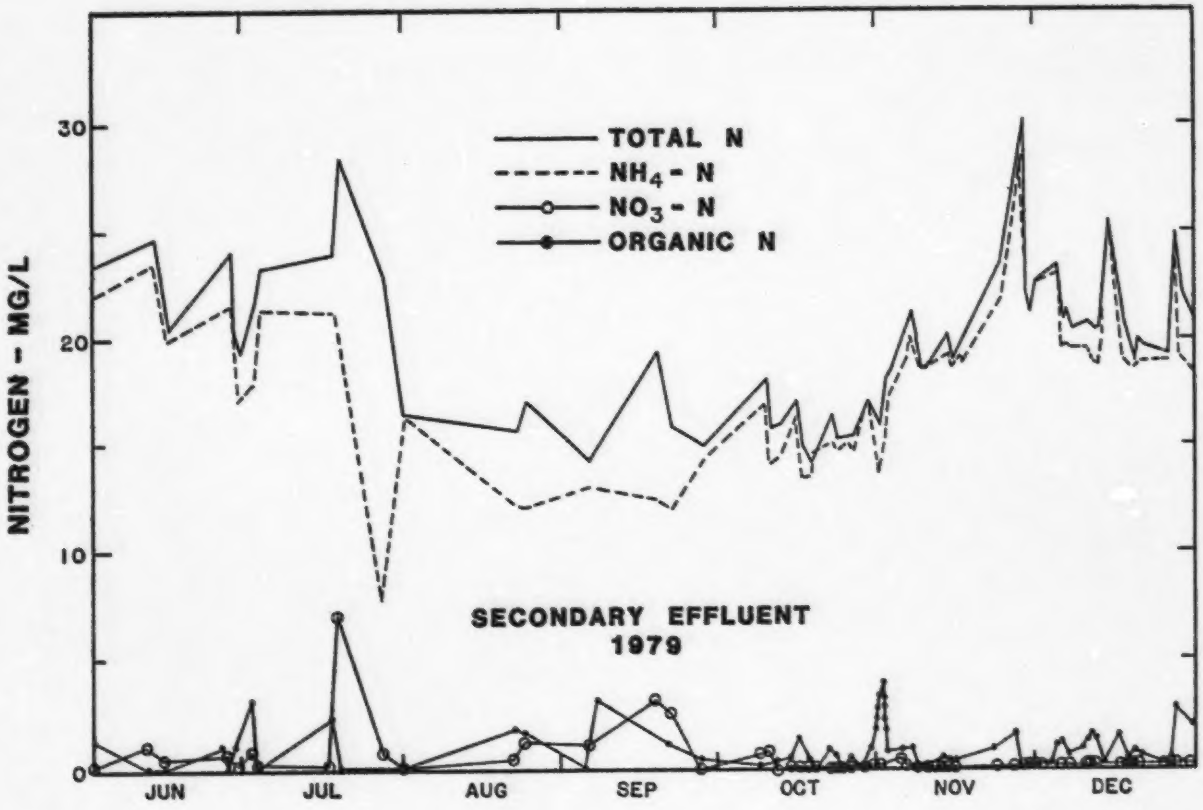

Figure 49. Nitrogen concentrations in secondary effluent entering basins, 1979.

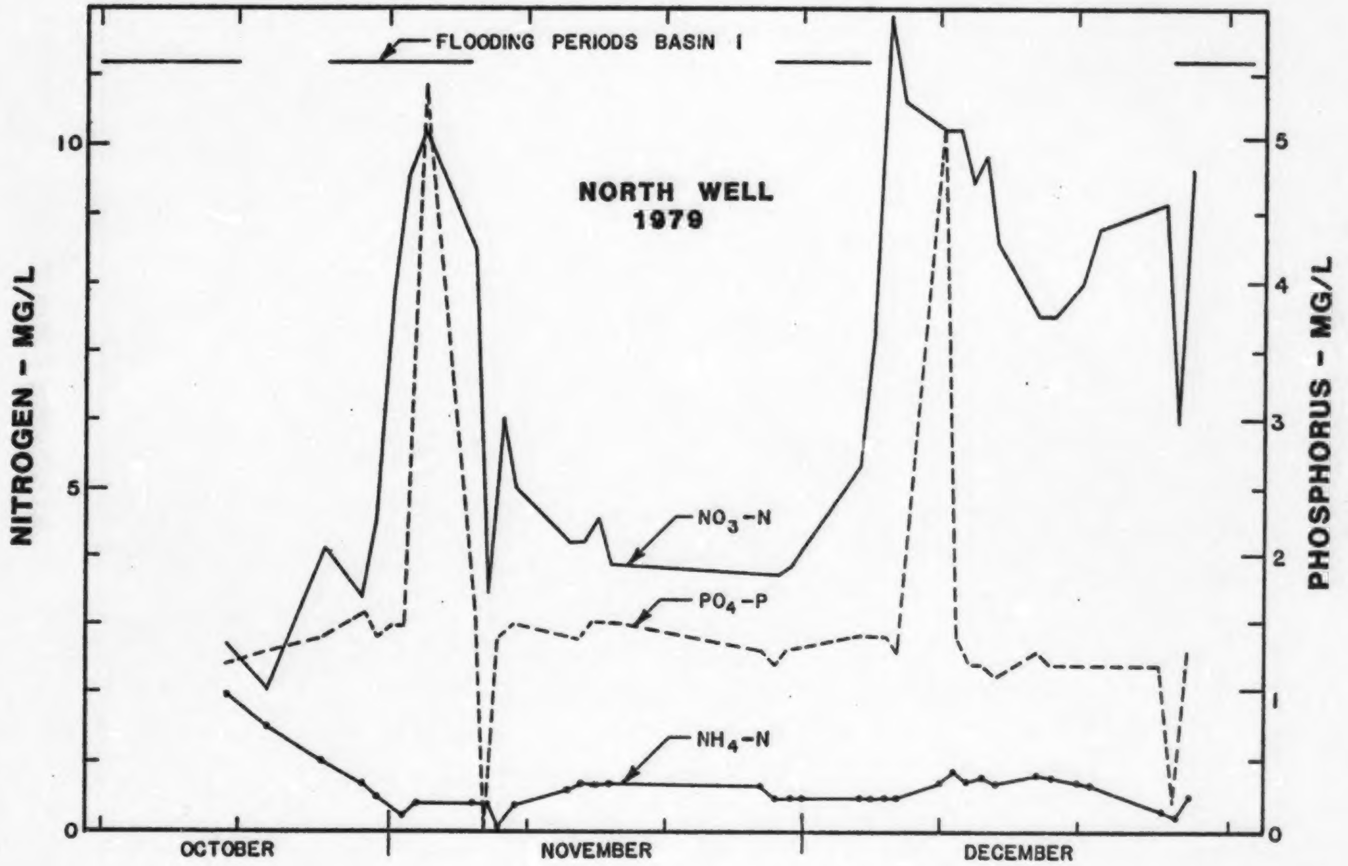

Figure 50. NO_3-N, NH_4-N, and PO_4-P concentrations in renovated water from the North Well, 1979.

the NO_3-N peaks would have been 28 days. This agrees with the actual time period of 31 days between the start of the 26 October and 27 November flooding periods. Thus, it is concluded that the NO_3-N peaks in November and December of Figure 50 corresponded to the flooding periods starting 26 October and 27 November, respectively.

The travel time of the NO_3-N peak of seven days at an infiltration rate of 0.3 m/day is quite small, and it implies a minimum average macroscopic velocity of 21/7 = 3 m/day (assuming direct vertical flow from the infiltration basin to the intake of the well, or a distance of 21 m). The water table in the November-December period was about 12 m below the bottom of the basins. Thus, the water had to travel through an unsaturated zone of 12 m and then an additional 9 m through the aquifer before reaching the bottom of the North Well. If Θ represents the volumetric water content of the vadose zone, the downward velocity in this zone at an infiltration rate of 0.3 m/day is about $0.3/\Theta$ m/day. The time it takes to reach the 12-m depth water table then is equal to $40\ \Theta$ days. Assuming that the water content in the aquifer is twice the water content in the vadose zone, the time for the water to travel the 9 m in the aquifer can similarly be calculated as $30 \times 2\ \Theta$ days. Since the total travel time is seven days ($40\Theta + 60\Theta = 7$), Θ is calculated as 0.07. According to this, the water content in the vadose zone would be 7% and that in the aquifer would be 14%. These figures could be reasonable, considering that the underground materials contained a lot of gravel and that the voids between the gravel particles were filled with sand. Assuming that the individual porosities of the gravel and the sand as such are both 40%, the porosity of the medium as a whole would be 40% of 40%, or 16%. This is close to the 14% calculated above.

The NH_4-N content of the renovated water from the North Well in 1979 was below 1 mg/ℓ and averaged 0.62 mg/ℓ (Figure 50). The average concentration of NO_2-N in the water was 0.045 mg/ℓ, and that for organic-N was 0.185 mg/ℓ.

For the Center Well water, NO_3-N contents in 1979 generally ranged between 5 and 8 mg/ℓ with an average of 6.33 mg/ℓ (Figure 51). NH_4-N concentrations were below 1 mg/ℓ and often close to zero. The average NH_4-N concentration was 0.12 mg/ℓ. Average concentrations of the other forms of N were 0.04 mg/ℓ for NO_2-N and 0.19 mg/ℓ for organic-N. Since the Center Well was perforated from 35 to 55 m, the water from the well was a mixture of renovated waters that have infiltrated at various parts in the basins and have had different times and distances of underground travel. Hence, individual NO_3-N peaks as in Figure 50 were not present and the NO_3-N values in Figure 51 were more uniform. Since the average total-N concentration in the effluent was about 18 mg/ℓ, the N removal was about 61%.

The NO_3-N concentrations in Center Well water were about the same as those in the North Well water. This and other similarities in chemical parameters (see for example, TOC, TDS, and fecal coliforms) indicate that most if not all of the water pumped from the Center Well was renovated sewage effluent and not native groundwater. The PO_4-P content of the

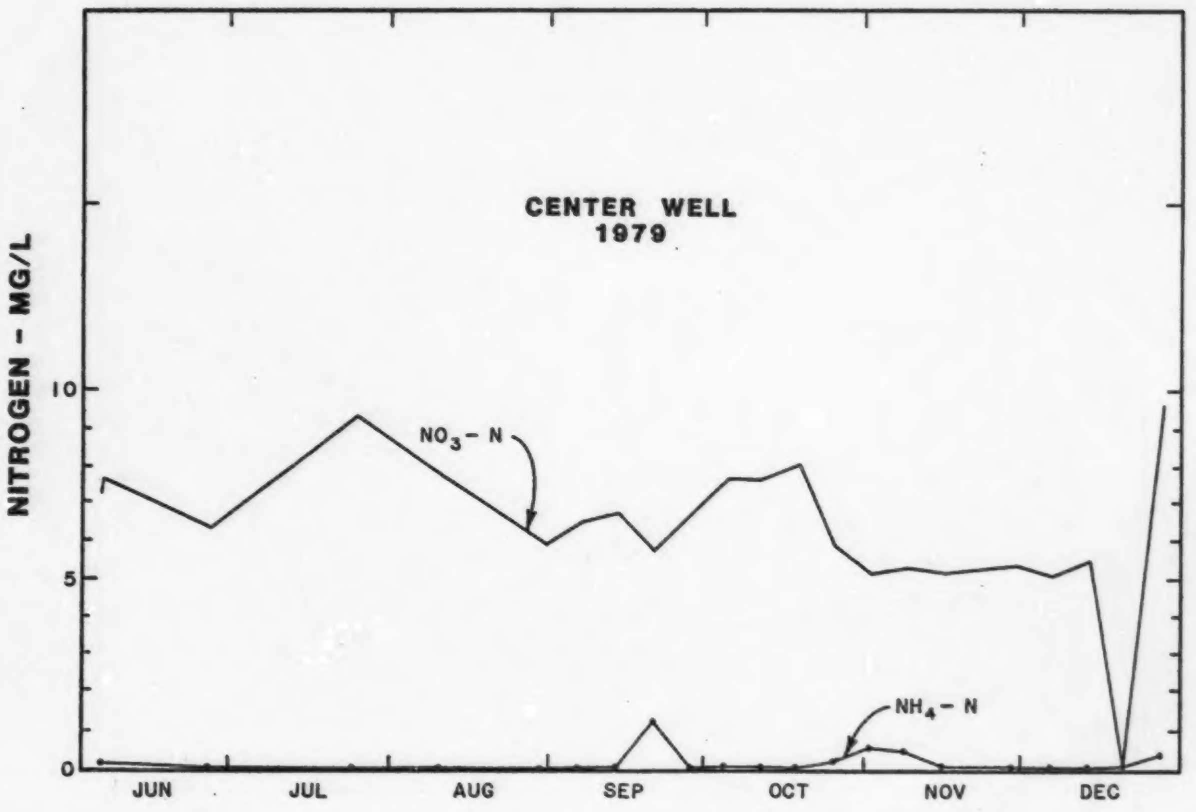

Figure 51. NO_3-N and NH_4-N concentrations in renovated water from the Center Well, 1979.

Center Well water was lower, than that in the renovated water from the shallow wells, but this could be due to additional immobilization of PO_4-P as the renovated water moved deeper into the aquifer.

In 1980, the total-N content of the secondary sewage effluent entering the infiltration basins generally ranged between 15 and 25 mg/ℓ (Figure 52) with an average of 17.4 mg/ℓ, again mostly as NH_4-N (16.1 mg/ℓ). The organic-N content averaged 1.3 mg/ℓ, and the NO_3-N content averaged 0.01 mg/ℓ. Renovated water from the North Well again showed several NO_3-N peaks (Figure 53). The peak on 10 May occurred 10 days after the 1 May beginning of a flooding period for basins 1 and 4. Since the average infiltration rate for basin 1 for that flooding period was 0.7 m/day (Figure 22), the arrival of the NO_3-N peak was about three times slower than would have been expected from the arrival times and infiltration rates associated with the November and December NO_3-N peaks in 1979. This difference was probably due to the long dry period (about two months) preceding the 1-10 May flooding period, which gave adequate time for the water to drain from the vadose zone. Thus, when flooding was started on 1 May, the vadose zone was relatively dry and it took the wet front longer to reach the groundwater table.

NO_3-N peaks were also observed in the renovated water from the 18-m Well (Figure 54). The peak in September occurred only a few days after the start of the September flooding periods for basins 2 and 3. This was due to the high infiltration rate in basins 2 and 3 after they were ripped (see Section 5). The NO_3-N concentration in the renovated water from the Center Well showed a decreasing trend (Figure 55) and averaged about 7 mg/ℓ. NH_4-N concentrations continued to be close to zero. Since the average total-N content of the secondary effluent was about 18 mg/ℓ Figure 52), the N removal in the soil aquifer system in 1980 was on the order of 60%. Average concentrations of the various forms of N in the renovated water during 1980 were as follows (in mg/ℓ):

	North Well	Center Well	18-m Well
NO_2-N	0.017	0.0	0.024
NO_3-N	7.5	7.1	8.3
NH_4-N	0.48	0.04	1.4
Organic-N	0.08	0.09	0.24
Total-N	8.1	7.2	9.9

The highest concentration of organic-N was observed in the 18-m Well which is in the center of the basin area and penetrates the aquifer the smallest distance. The studies on NO_3-N peaks and coliform peaks in the water from the 18-m Well have indicated that the effluent water moves very fast from the infiltration basin to the intake of the 18-m Well. Hence, "treatment" is reduced and the renovated water from the 18-m Well has more organic carbon, fecal coliforms, and other substances than that from the other wells.

The N concentrations in the renovated water were sufficiently close to the "no-problems" range of 0-5 mg/ℓ (Ayers, 1975) to permit essentially unrestricted irrigation of crops.

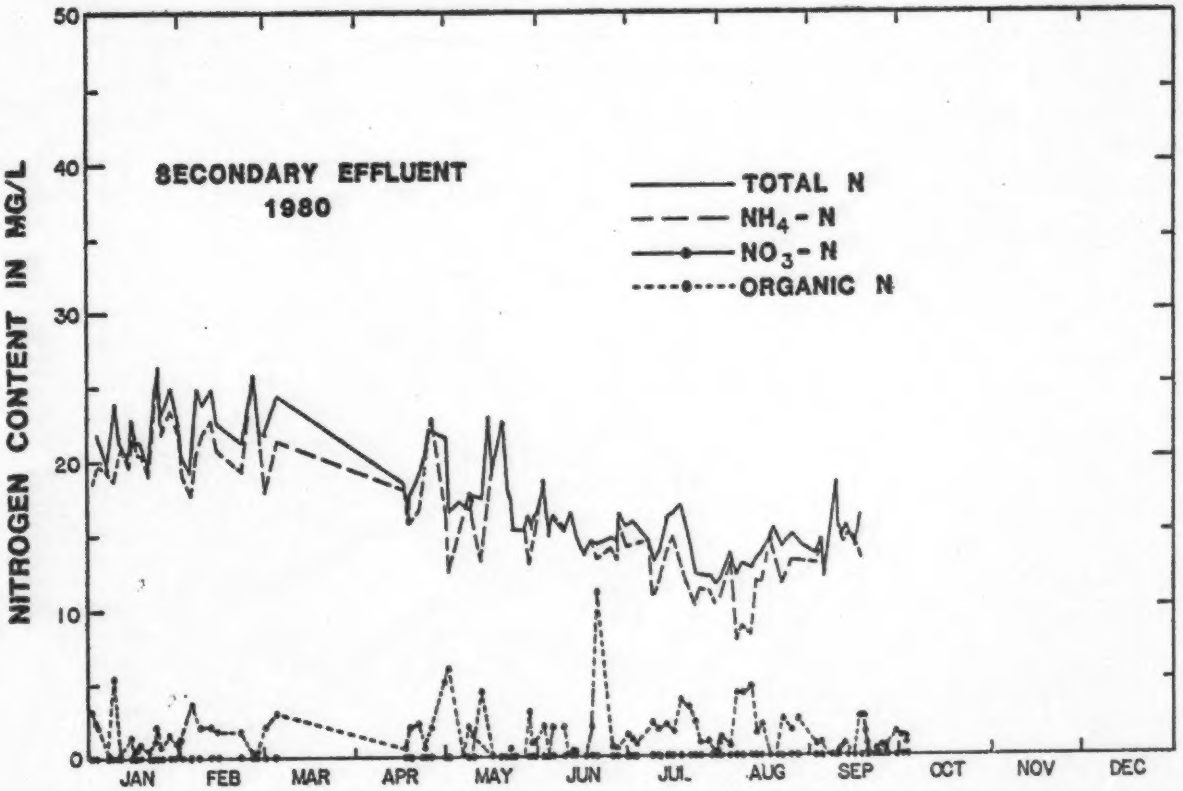

Figure 52. NO$_3$-N, NH$_4$-N, organic-N, and total-N concentrations in secondary sewage effluent, 1980.

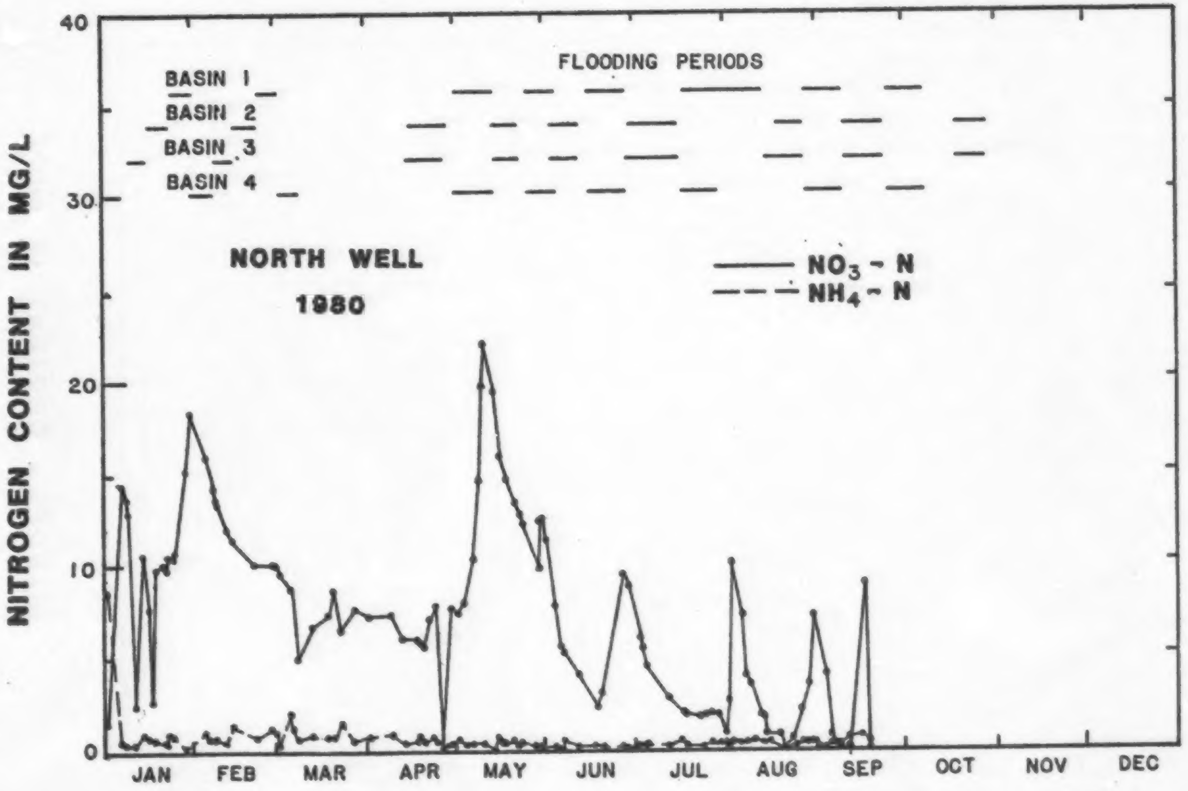

Figure 53. NO3-N and NH4-N concentrations in renovated water from the North Well, 1980.

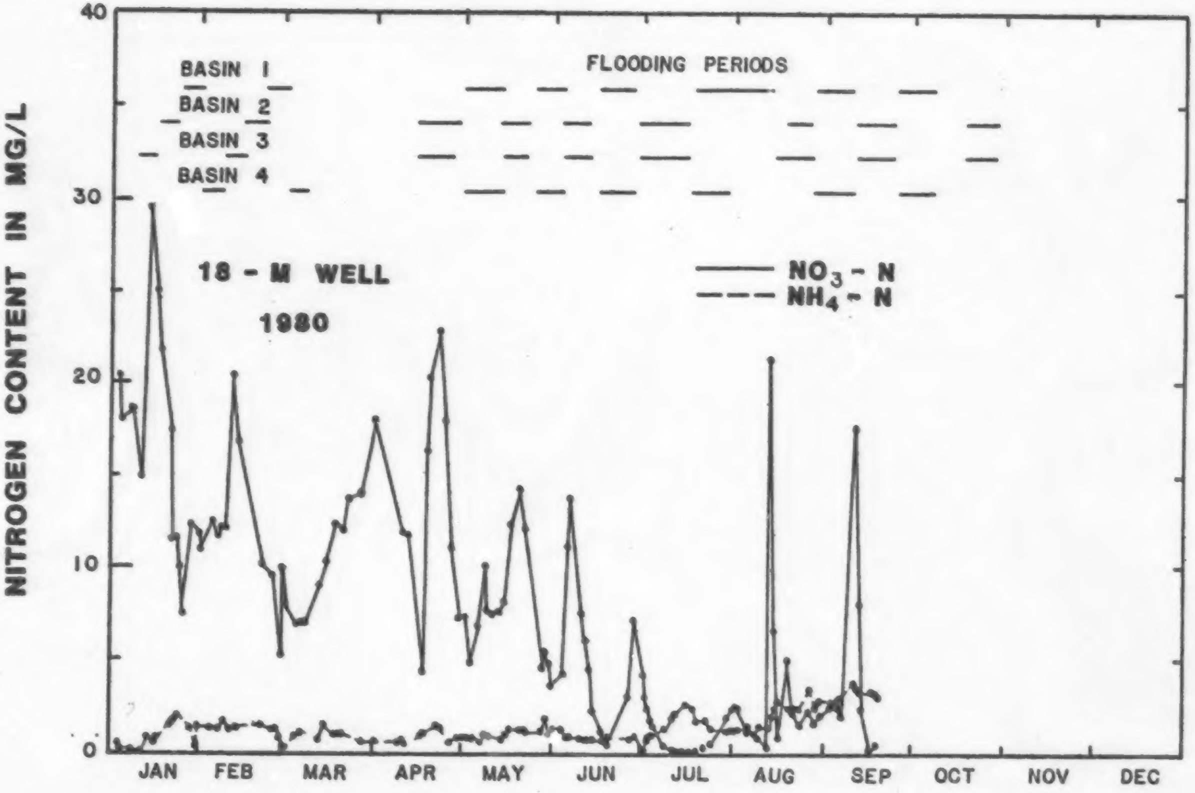

Figure 54. NO_3-N and NH_4-N concentrations in renovated water from the 18-m Well, 1980.

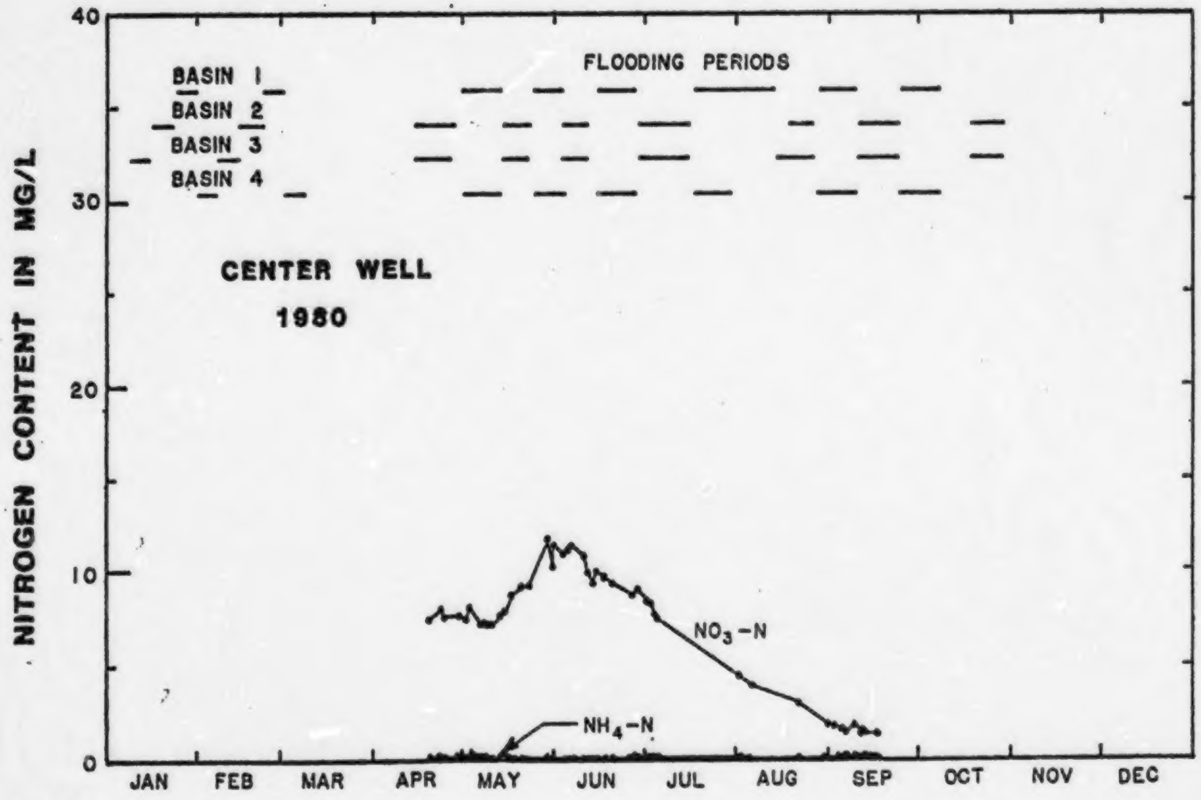

Figure 55. NO_3-N and NH_4-N concentrations in renovated water from the Center Well, 1980.

Nitrogen Transformations and Denitrification Rates

Since denitrification is by far the most important mechanism for N removal in rapid infiltration systems, detailed studies were made of the rates of N transformation in the soil of the infiltration basins at the 23rd Avenue Project. The studies were carried out in the winter of 1979 and the summer of 1980, and consisted of laboratory studies of N transformations in soil samples from the infiltration basins. Flooding and drying periods of the basins were nine days each and water depths were about 20 cm. Soil samples were collected only during drying periods. Subsamples were extracted and analyzed, as soon as possible, for inorganic N components with a Technicon AutoAnalyzer I. Other subsamples were prepared for measuring the rates of denitrification and moisture contents.

The acetylene inhibition method was used to quantitatively measure denitrification rates. The method blocks biological reduction of nitrous oxide to molecular N. Thus, denitrification rates can be measured by production of nitrous oxides, which are easier to detect than the omnipresent molecular N. Soil subsamples were placed in 40-mℓ septum vials and treated with about 90 μg NO_3-N/g, since nitrification was inhibited by acetylene (0.1 atm). Head space samples of 1.0 mℓ were obtained after incubation at 28 C for 24 hours and were analyzed for nitrous oxide with a gas chromatograph equipped with a thermal conductivity detector. Control soil samples without nitrate-N additions were also analyzed. The atmosphere in the vials was either helium or air with about 0.1 atm of acetylene.

The results showed that denitrification rates were greatest during the first two to three days of drying regardless of the season of the year (Table 5). Soil environments promoted lower denitrification rates throughout the dry period in the summer than in the winter. The higher summer temperature (35-40°C) and faster drying conditions rapidly created an oxidized environment unfavorable for denitrification in the surface of the soil basins. Soils treated with helium plus nitrate-N had the greatest rates of denitrification, followed by soils treated with air plus nitrate and air only, respectively. The additions of NO_3-N increased denitrification for only two to three days in soils treated with air. On the first day of drying, the low rates of denitrification in soil treated with air only resulted from the low amount of nitrified-N in the soil sample and the inhibition of nitrification by acetylene. With nitrification inhibited, denitrification could not occur, so NO_3-N was added to soils treated with air and helium. The NO_3-N treatments were the most accurate estimates of denitrification rates during drying in the surface soil (0-5 cm) of an infiltration basin.

The results indicated that conditions necessary for maximum denitrification were present in the surface of the soil basins for only the first 48 to 72 hours of the dry period during summer or winter. The most important factors limiting denitrification were the rates of nitrification that controlled the concentration of NO_2-N and/or NO_3-N and the amount of the available organic carbon.

TABLE 5. NITROGEN TRANSFORMATION AND DENITRIFICATION RATES IN SOIL BASINS INTERMITTENTLY FLOODED WITH SECONDARY SEWAGE EFFLUENT FROM 23RD AVENUE TREATMENT PLANT *

Days	Soil Nitrogen†			Denitrification rates ††			Water
Dry	Ammonium-N		Nitrified-N	He + NO_3-N	Air + NO_3-N	Air (Only)	Content
	(μgN/g)			(μgN_2O-N evolved/g/day)			(%)
			Winter:	Oct - Nov - Dec 1979			
1	110	(111)	1	63	19	<1	24.0
2	91	(127)	28	58	23	11	21.9
3	64	(121)	57	78	9	8	15.6
7	33	(155)	122	74	8	5	11.8
10	47	(198)	151	96	11	8	9.8
			Summer:	Jul - Aug - Sep 1980			
1	170	(176)	6	23	23	<1	29.6
2	81	(128)	47	33	14	7	21.4
3	45	(147)	102	25	4	3	18.0
7	28	(186)	158	15	2	1	8.6
10	20	(208)	188	16	1	1	6.8

* Results are data averaged from 3 and 5 dry periods during winter and summer, respectively.

† Soil samples were collected, extracted and analyzed as soon as possible for NH_4-N, NO_2-N and NO_3-N. Nitrified-N was the $\mu gNO_2\&NO_3$-N/g. The parentheses indicate the total inorganic nitrogen.

†† Denitrification rates were measured in soil samples collected during dry periods with the acetylene inhibition method. Soil samples were incubated for 24 hours in flasks containing helium or air and 0.1 atm of acetylene. Nitrate N additions were equivalent to 91 μgNO_3-N/g of soil.

The summation of denitrification rates during 10 days of drying accounted for 21% N removal during winter and 9% during summer. These results are based on an average infiltration rate of 30 cm/day and NH_4-N concentrations in the effluent of 15 mg/ℓ in winter and 12 mg/ℓ in summer. N concentrations in effluent and renovated water indicated, however, that about 60% of the N was removed. Thus, the technique accounted for only a fraction of the N removal and additional denitrification must take place at greater depth and/or during flooding.

The N transformation in the soil basins flooded with secondary effluent showed a natural and expected conversion of NH_4-N to nitrified-N (Table 5). The nitrification rates in the summer were about twice those during the winter. This, again, was the result of higher summer temperatures that enhanced the oxidizing metabolic activity of nitrifying bacteria. The major factor limiting denitrification was the concentration of NO_2-N or NO_3-N produced by nitrification during the first 48 to 72 hours of the dry period. The carbon energy needed for denitrification was also an important limiting factor for the secondary effluent with its low concentration of organic carbon.

The results indicated that management practices for maximizing denitrification in the surface soil of rapid-infiltration systems (in terms of percent N-removal) should induce a reduction in the amount of N applied, enhancement of the environment favorable for the nitrification-denitrification reaction, an increase in the numbers of physiologically active denitrifying bacteria, or any combination of these practices. A reduction in the amount of N applied could be attained by reducing infiltration rates to about 15 cm/day or less. This should increase N-removal to an estimated 60 to 80%. With this approach, rates of denitrification would not change, only the amount of N applied. Enhancement of the environment favorable for nitrification-denitrification reactions during drying could be achieved by maintaining the soil surface near saturation with intermittent daily sprinkler application of small quantities of sewage effluent. This would result in little NO_3-N movement and induce extended activity of denitrification in reduced microsites. Laboratory studies on soil columns, however, failed to show any major increases in N removal by this process. The numbers of physiologically active denitrifying bacteria in the surface could be increased by timely additions of organic carbon to support their growth and denitrifying activity. The denitrifying population should be physiologically active to provide a denitrification rate rapid enough for complete N-removal. Extra organic carbon could be applied to the soil surface by adding carbon to the sewage effluent during the last two days of flooding and/or by sprinkler application of sewage effluent during drying. Combinations of the above practices would be necessary to maintain N-removals of 80% or more at infiltration rates of 20 to 60 cm/day.

PHOSPHORUS

PO_4-P concentrations in the secondary effluent entering the infiltration basins from the 32-ha lagoon during 1975 and 1976 (the pre-bypass-channel period) generally were between 4 and 10 mg/ℓ (Figure 56)

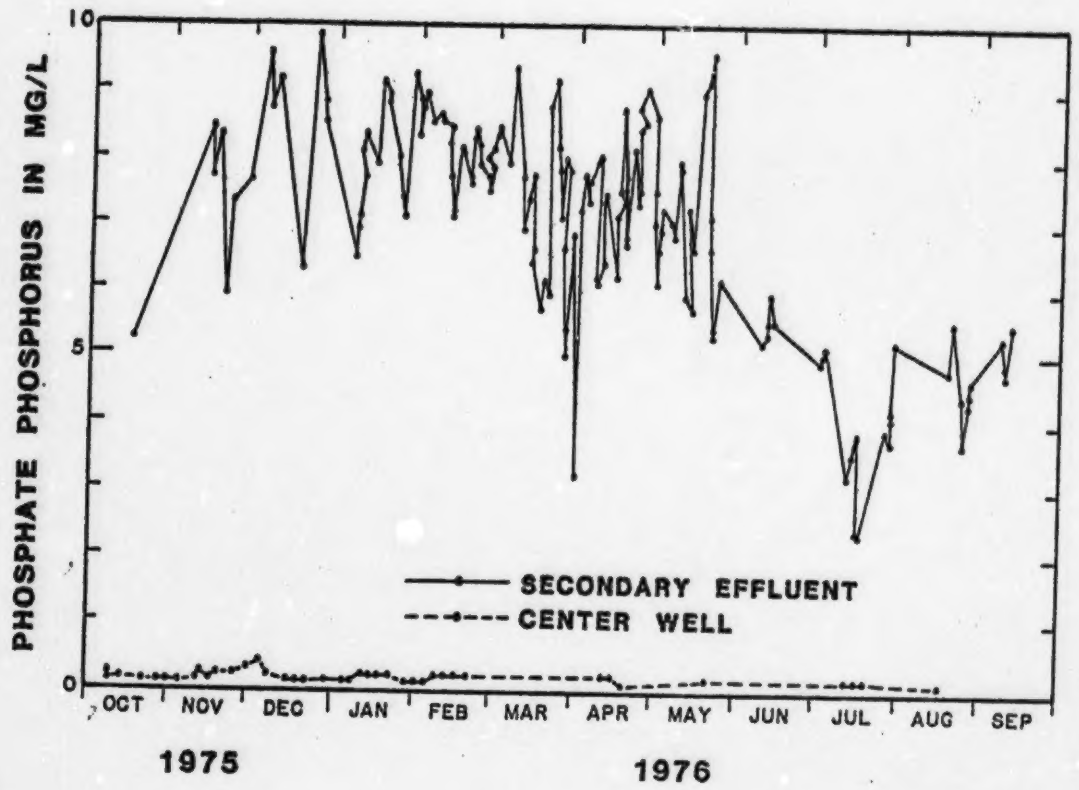

Figure 56. PO_4-P concentrations of secondary effluent entering infiltration basins and of renovated water from the Center Well, 1975-1976.

and averaged about 7 mg/ℓ. PO_4-P concentrations of the renovated water remained fairly constant at about 0.2 mg/ℓ (Figure 56). In 1979 (after construction of the bypass channel), PO_4-P concentrations in the secondary effluent generally ranged between 2 and 12 mg/ℓ and averaged about 6 mg/ℓ (Figure 57). PO_4-P concentrations in the renovated water from the North Well generally were between 1 and 1.5 mg/ℓ (Figure 50) and averaged 1.5 mg/ℓ. Interestingly, this renovated water exhibited PO_4-P peaks that coincided with the NO_3-N peaks and, hence, were associated with effluent water that infiltrated at the beginning of a flooding period. The 1979 PO_4-P concentrations in the water from the Center Well were more uniform (Figure 57) and averaged 0.32 mg/ℓ. This is almost 80% lower than the average PO_4-P concentrations in renovated water from the North Well, indicating additional precipitation of PO_4-P deeper in the aquifer.

In 1980, PO_4-P concentrations averaged 5.41 mg/ℓ for the secondary effluent, 2.71 mg/ℓ for the renovated water from the 18-m Well, 1.34 mg/ℓ for that from the North Well, and 0.17 mg/ℓ for that from the Center Well (Figure 58). The renovated-water curves in Figure 58 show the additional decrease of the PO_4-P concentration as the renovated water continues to travel laterally (as to the North Well, for example) or vertically (as to the Center Well). The curves for the renovated water also exhibited a gradually increasing trend, especially after June 1980 when infiltration rates in the basins were markedly increased due to ripping of the soil. Thus, PO_4-P removal in the soil-aquifer system also depended on the infiltration rate.

FLUORIDE

Fluoride concentrations in 1976 averaged 1.22 mg/ℓ in the effluent from the 32-ha lagoon and 0.7 mg/ℓ in the renovated water from the Center Well. Thus, fluoride concentrations in the renovated water were below maximum limits for drinking or unrestricted irrigation.

BORON

Boron concentrations in 1976 were 0.5 mg/ℓ in effluent and renovated water. This is below the maximum allowable concentration in irrigation water for even the most boron sensitive crops.

MICROORGANISMS

Fecal Coliform Bacteria

Fecal coliform concentrations in 1975 and 1976 were on the order of 10^6/100 mℓ for the unchlorinated secondary effluent as it entered the 32-ha lagoon, and 10^4/100 mℓ as it left the 32-ha lagoon to enter the infiltration basins (Figure 59). Renovated water was essentially free from fecal coliforms, except after new flooding periods were started in the two

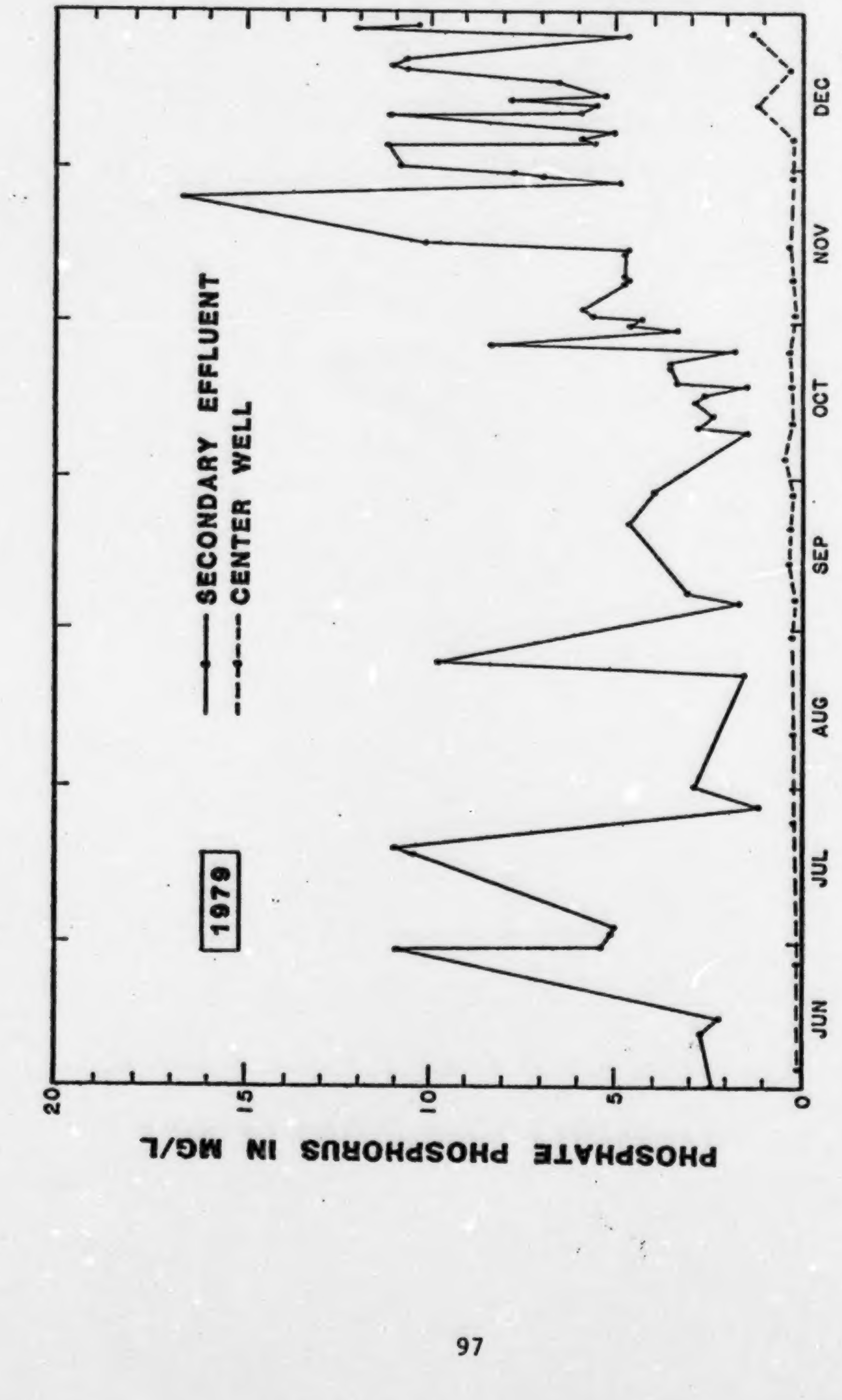

Figure 57. PO4-P concentrations of secondary effluent entering infiltration basins and of renovated water from the Center Well, 1979.

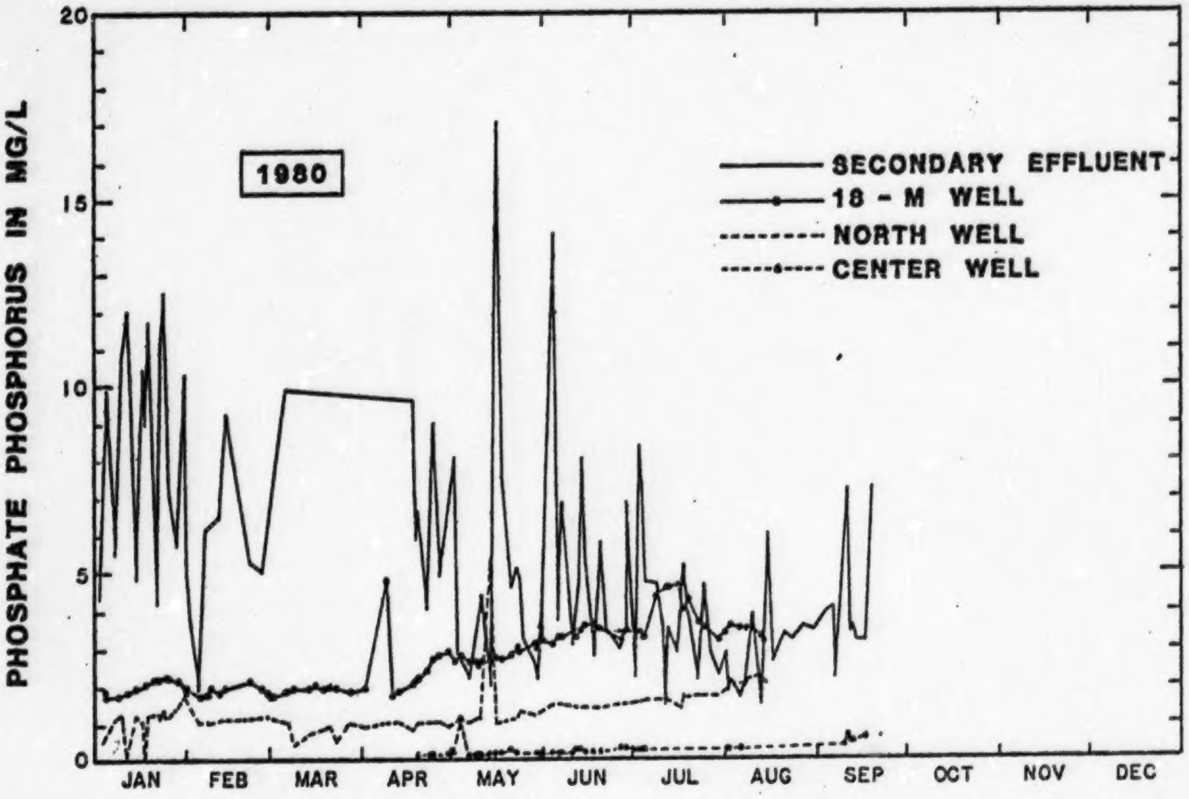

Figure 58. PO_4-P concentrations of secondary effluent and of renovated water from the North, Center, and 18-m Wells, 1980.

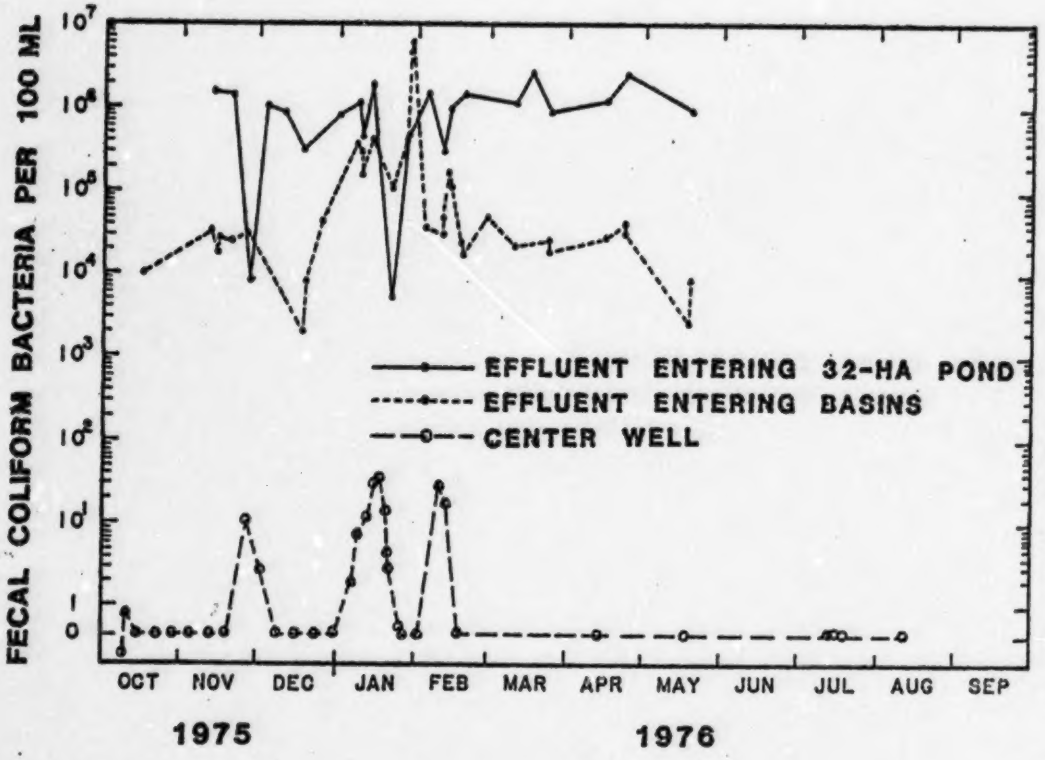

Figure 59. Fecal coliform concentrations in secondary effluent entering and leaving the 32-ha lagoon, and in the renovated water from the Center Well, 1975-1976.

basins adjacent to the Center Well (Figure 59). At those times, short-term increases of the fecal coliform concentration to a maximum of about 40/100 mℓ were observed. Similar results were obtained in 1979, except that the peak concentrations did not exceed 10/100 mℓ (Figure 60). Average fecal-coliform concentrations in 1979 were 1.25/100 mℓ for the renovated water from the North Well, and 2.3/100 mℓ for that from the Center Well.

In 1980, the average fecal coliform concentration of the sewage effluent entering the infiltration basins was 1.8 x 10^6/100 mℓ. Fecal coliform concentrations of the renovated water in 1980 were considerably higher than in previous years. Renovated water from the North Well had peak coliform concentrations of 43 and 45/100 mℓ (Figure 61), but the average concentration was 3.7/100 mℓ. The Center-Well water showed peak coliform concentrations of 140 and 160/100 mℓ and had an average concentration of 22.1/100 mℓ (Figure 62). Very high coliform peaks were observed in the renovated water from the 18-m Well in the center of the project (Figure 63), where peaks regularly exceeded values of 1000/100 mℓ and at one time shot up to 17,000/100 mℓ. The two highest peaks occurred in August and September, after the infiltration basins had been ripped to increase infiltration rates. The average fecal coliform concentration for the water from the 18-m Well was 993/100 mℓ.

The average fecal coliform concentration of 22.1/100 mℓ for the "production" water from the Center Well was too high for unrestricted irrigation under the proposed new standards for effluent irrigation in Arizona, which specify a maximum of 2.2/100 mℓ for the geometric mean and of 25/100mℓ for a single sample. Lower fecal coliform concentrations in the renovated water could be obtained by flooding schedules that avoid simultaneous new flooding of basins 2 and 3. If basins 1 and 2 are flooded at the same time, and basins 3 and 4 for the next flooding sequence, the center wells would never pump renovated water that infiltrated in the adjacent basins 2 and 3 at the same time. If, for example, basins 1 and 2 are filled, the initial infiltration from basin 2 would increase the fecal coliform concentration in the water from the Center Well. However, the well would also pump renovated water that infiltrated at the end of the flooding period from basin 3. Since this renovated water would have a low coliform content, it would reduce the fecal coliform concentration in the pumped water. Also, the 23rd Avenue Treatment Plant will start chlorinating its effluent at the end of 1980, which should reduce coliform concentrations in the effluent and the renovated water. This will be studied after the chlorination has been in effect for some time. Preliminary results indicated that chlorination of the effluent reduced the fecal coliform concentration to a few thousand/100 mℓ for the effluent and to a range of 0-4/100 mℓ for the renovated water (average less than 2.2/100 mℓ).

The higher fecal coliform concentrations in the renovated water in 1980 must have been caused by the higher infiltration rates resulting from the use of the bypass channel, and by the ripping of the surface soil in the infiltration basins. The high values in the renovated water from the 18-m Well must also have been due to the very fast arrival of

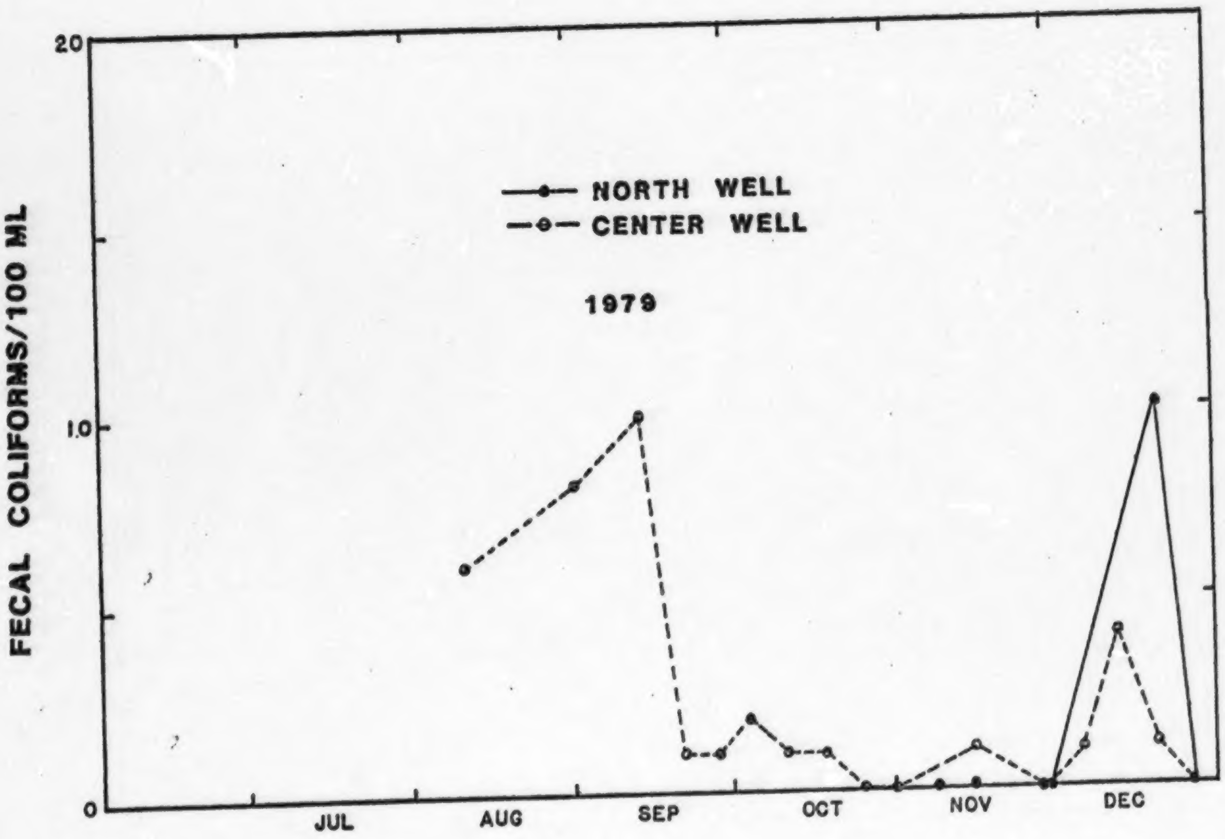

Figure 60. Fecal coliform concentrations in renovated water from the North and Center Wells, 1979.

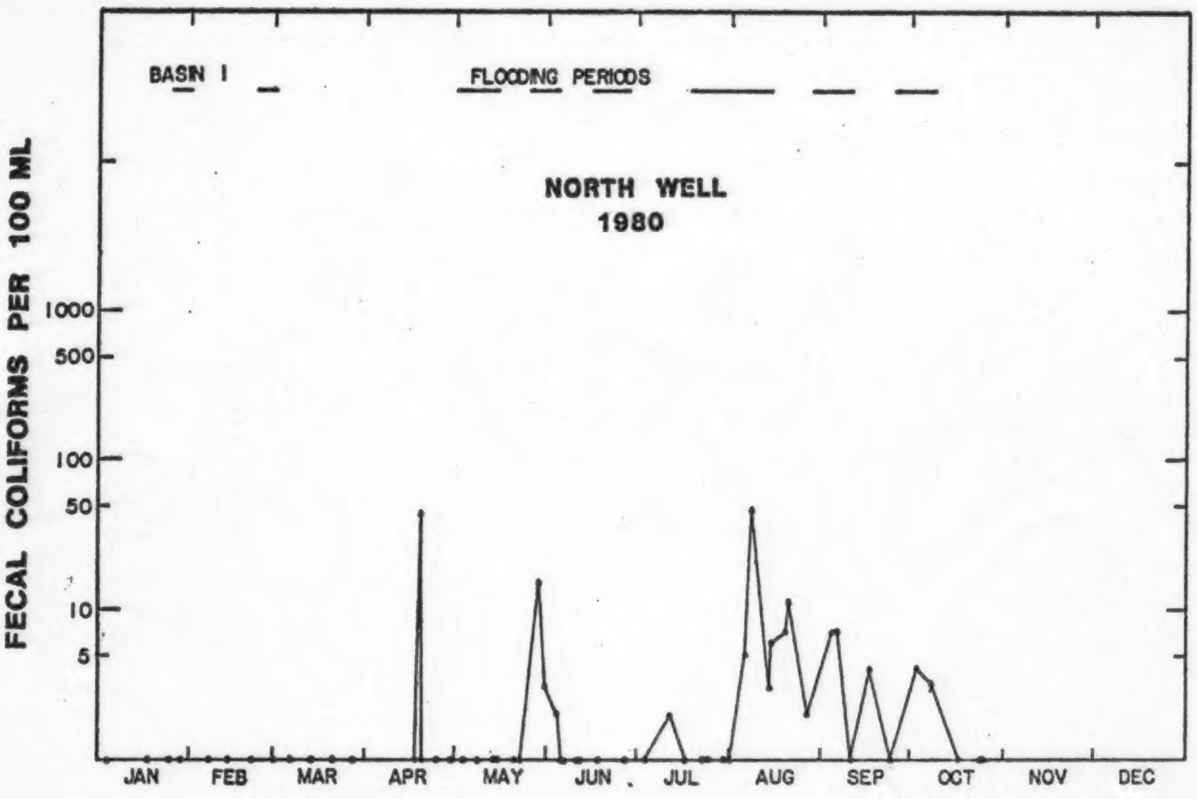

Figure 61. Fecal coliform concentrations in renovated water from the North Well, 1980.

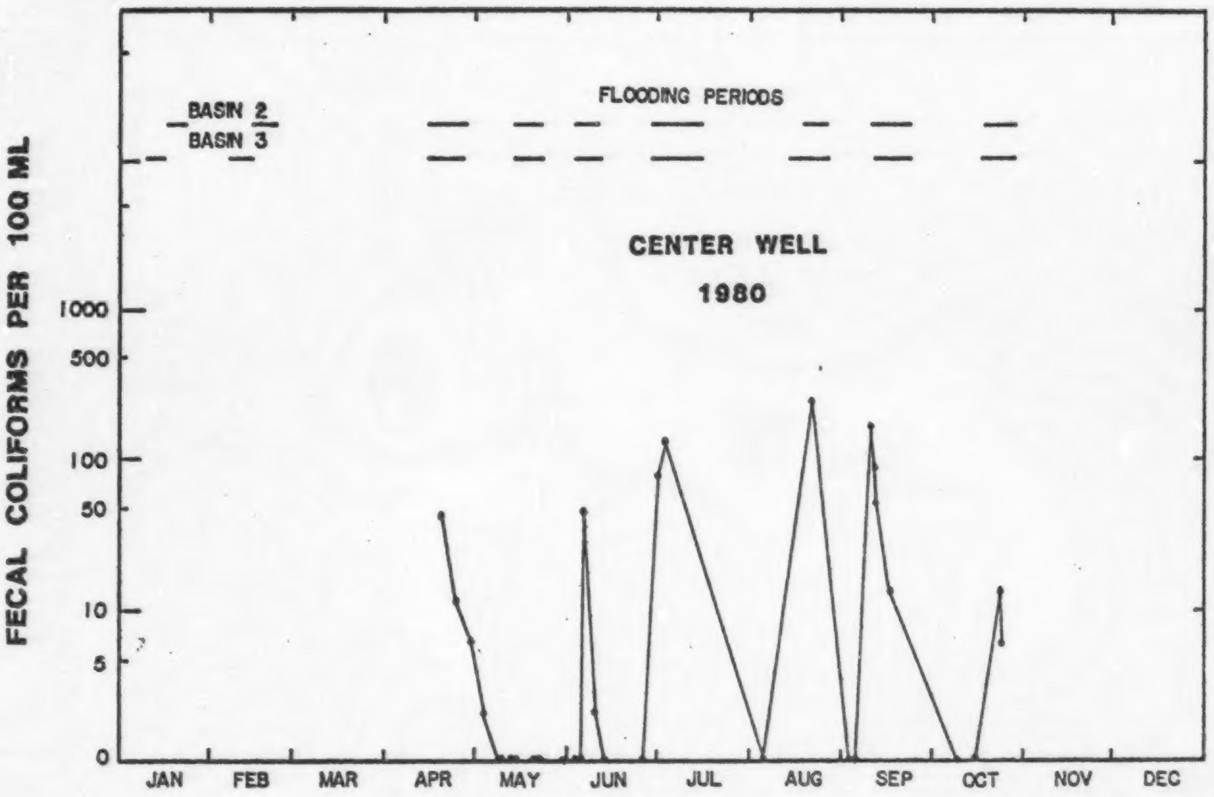

Figure 62. Fecal coliform concentrations in renovated water from the Center Well, 1980.

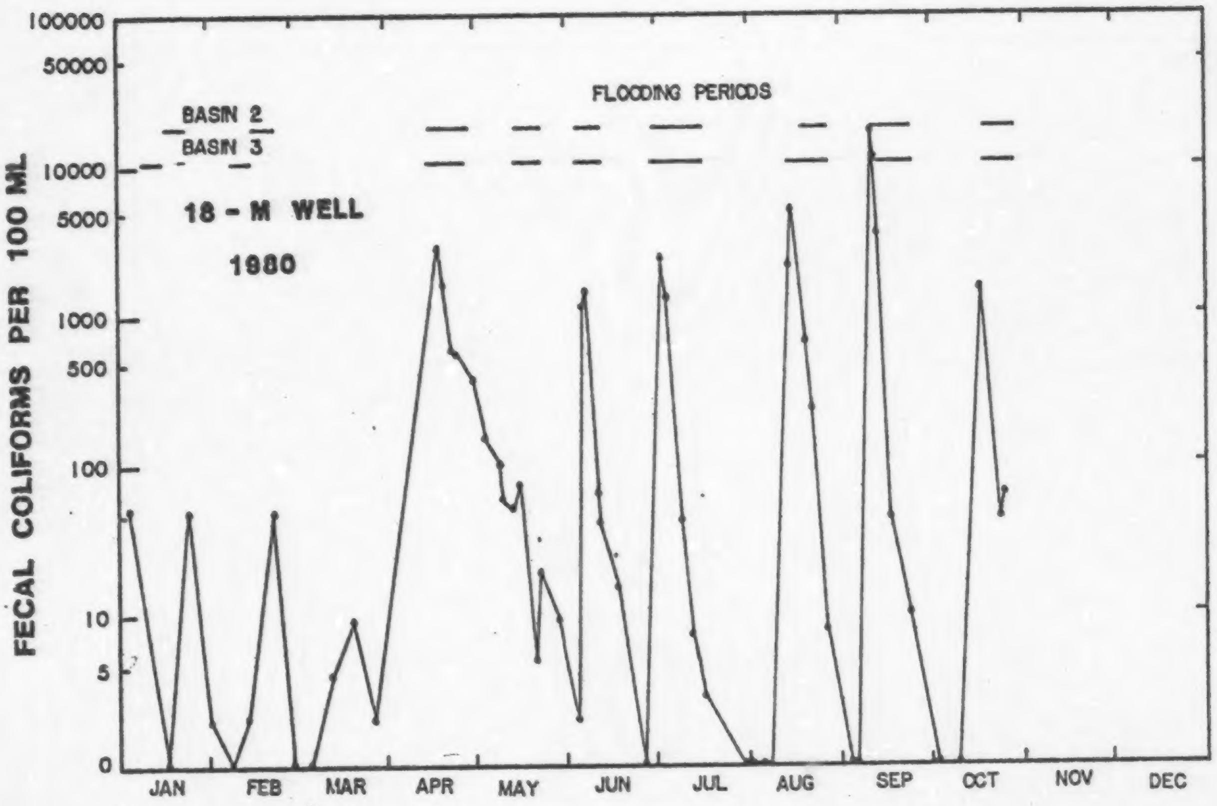

Figure 63. Fecal coliform concentrations in renovated water from the 18-m Well, 1980.

renovated water at this well from the infiltration basins, as indicated by the short times between starts of new flooding periods in basin 2 and the appearance of NO_3-N and coliform peaks in the renovated water from the 18-m Well (Figures 54 and 63).

Viruses

Renovated water from various wells was sampled twice in 1980 for viruses, pumping 800 to 2000 ℓ of renovated water through special filters to concentrate the viruses in a small volume that could be sent to the laboratory for assay. In January 1980, viruses were concentrated into small samples by pumping well water through a fiberglass cartridge filter and injecting hydrochloric acid to adjust the pH of the well water to 3.5 and aluminum chloride to a concentration of 0.0005 aluminum chloride before the well water entered the filter (Gilbert et al., 1976). The adsorbed viruses were eluted from the filters with glycine buffer (pH of 11.5) and the eluate was neutralized, frozen, and shipped to C. P. Gerba of the Baylor College of Medicine, Houston, Texas, for analysis. In June 1980, samples were concentrated by simply pumping well water through recently developed positively charged filters. The filters were packed in ice and shipped to Baylor College of Medicine where the viruses were eluted and assayed. Duplicate samples were obtained in all cases. Average virus concentrations are shown in Table 6.

An average of about 0.15 PFU/100 ℓ were detected on the January sampling date while about 1 PFU/100 ℓ was found in June (Table 6). All of the virus concentrations were well below the 1 PFU/40 ℓ limit proposed by the Arizona Department of Health Services for wastewater to be used for unrestricted irrigation.

The differences in virus concentrations for the two sampling dates could have been due to differences in the concentration techniques, to seasonal variation in the virus concentrations in the sewage, or to differences in virus survival times in the soil during winter and summer.

When wells at the Flushing Meadows Project were sampled every two months for the entire year of 1974, no viruses were detected in the renovated water (Gilbert et al., 1976). However, the maximum volume of water concentrated from the Flushing Meadows wells was 400 ℓ, while most of the samples from the 23rd Avenue wells represented at least 1200 ℓ. Also, the top m of soil at the Flushing Meadows Project was finer than that at the 23rd Avenue Project. On the other hand, the samples at the 23rd Avenue Project were taken from greater depth than those at the Flushing Meadows Project. Soil column studies and analysis of soil samples from recharge basins have shown that most of the viruses are adsorbed near the surface (Lance and Gerba 1976, Hurst et al. 1980). Much of the textural differences in the surface layers of the recharge basins are due to the removal of the finer surface material for construction of the dikes around the basins at the 23rd Avenue site. Once effluent water had penetrated the surface layers and entered the underlying strata of coarse sands, gravel and cobbles, it is possible that some water moved primarily through large pores and channels where virus adsorption would be limited.

TABLE 6. VIRUS CONCENTRATIONS AND ISOLATES FROM CONCENTRATED RENOVATED WATER SAMPLES OF THE 23rd AVENUE PROJECT

Sampling date	Well	Sample size ℓ	Virus Count (PFU/100 ℓ)	Viruses isolated
2 Jan. 1980	North	800, 2000	0.1	Coxsackie-B3
	18-m	1800, 400	0.2	Coxsackie-B3
10 Jun. 1980	North	1200, 1200	1.5	Coxsackie-B4
	18-m	1200, 1200	2.0	Polio-1, Coxsackie-B3
	Center	1200, 1200	1.3	Polio-1

Soil column studies showed that in coarse sands, viruses penetrated below a depth of 2.4 m when the infiltration rate exceeded 1 m/day (Lance and Gerba 1980). Although the average infiltration rate at the 23rd Avenue site was below 0.3 m/day during the periods when the wells were sampled, the infiltration rate during the first day of flooding could have been as high as 1 m/day. If viruses moved through the surface during the first day, they could survive for some time under the anaerobic conditions present at lower depths. Field and laboratory studies showed that viruses survive much longer under anaerobic conditions than under aerobic conditions (Hurst et al., 1980).

More virus samples will be taken to study the effect of higher infiltration rates and chlorination on virus movement.

ORGANIC CARBON

In 1979, the concentrations of total organic carbon (TOC) in the secondary effluent entering the infiltration basins generally ranged between 8 and 15 mg/ℓ (Figure 64) and averaged 11.2 mg/ℓ. In the renovated water, TOC concentrations were more uniform and averaged 2.4 mg/ℓ for water from the North Well and 2.0 mg/ℓ for water from the Center Well. TOC concentrations in the secondary effluent and renovated water from various wells in 1980 are shown in Figures 65, 66, and 67. The average values were 9.7 mg/ℓ for the secondary effluent, 2.2 mg/ℓ for the renovated water from the North Well, 1.7 mg/ℓ for that from the Center Well, and 3.3 mg/ℓ for that from the 18-m Well. The 18-m Well, which is next to basin 2 and penetrates the groundwater the shortest distance, thus showed the highest TOC concentration of the renovated water samples. The additional movement in the aquifer (mostly lateral to the North Well and vertical to the perforated section of the Center Well) thus produced more removal of organic carbon.

Previous work at the Flushing Meadows Project (Bouwer et al, 1980) showed that the BOD of the renovated water was essentially zero, indicating that most of the organic carbon in the renovated water must be associated with organic compounds that are not readily degraded in the soil and aquifer. Some of the organic carbon undoubtedly is in the form of humic and fulvic acids. However, trihalomethanes and other organohalides, plus plasticizers, pesticides, and other compounds entering the sewage effluent via industrial wastes or from chemical products used in the household could be present. Chlorinated organics may also be formed by reactions between chlorine and organic compounds when raw water is treated for municipal water supply, and again when sewage effluent is chlorinated in the sewage treatment plant. The presence and identity of trace organics in renovated water is a major factor of concern in the potable reuse of such water, because the organics can include known or suspected carcinogens for man or animals.

The identity of the trace organics in the secondary effluent and in the renovated water from various wells was studied in the first part of

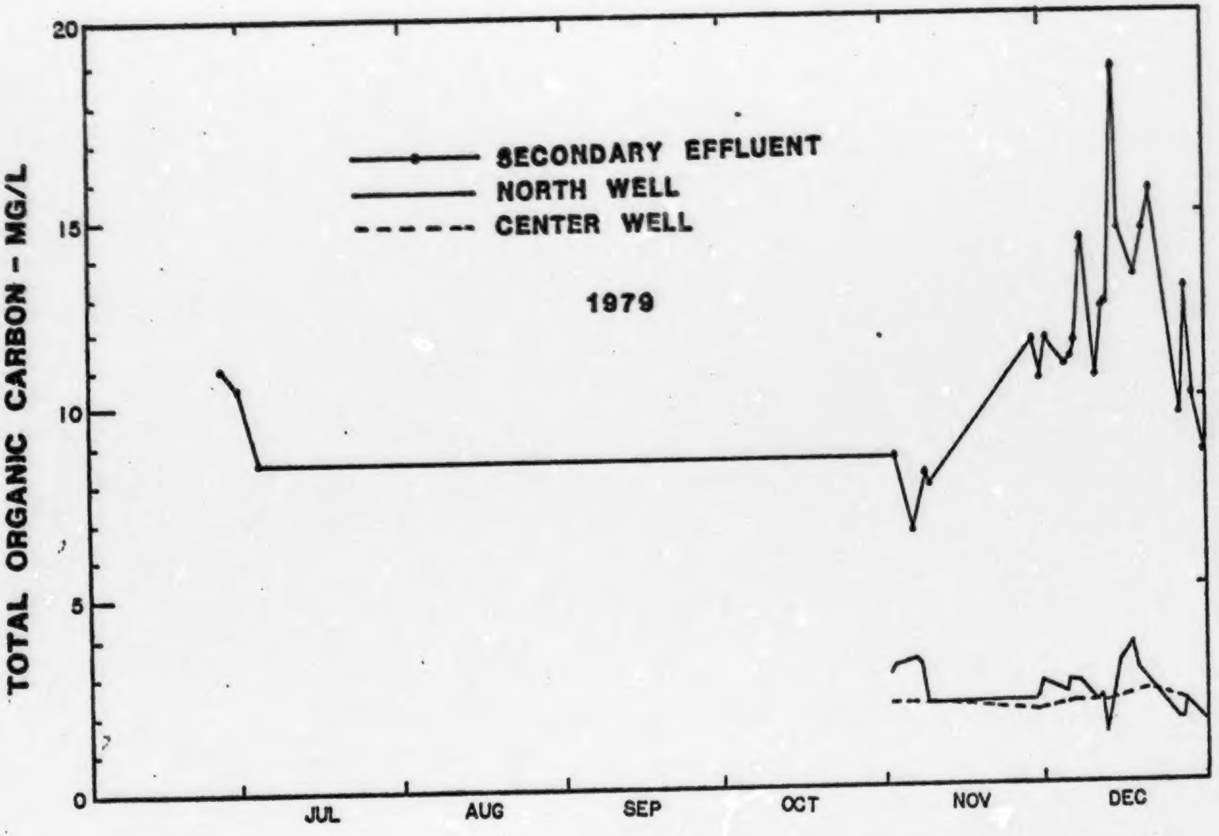

Figure 64. TOC concentrations in secondary effluent and in renovated water from the North Well and Center Well, 1979.

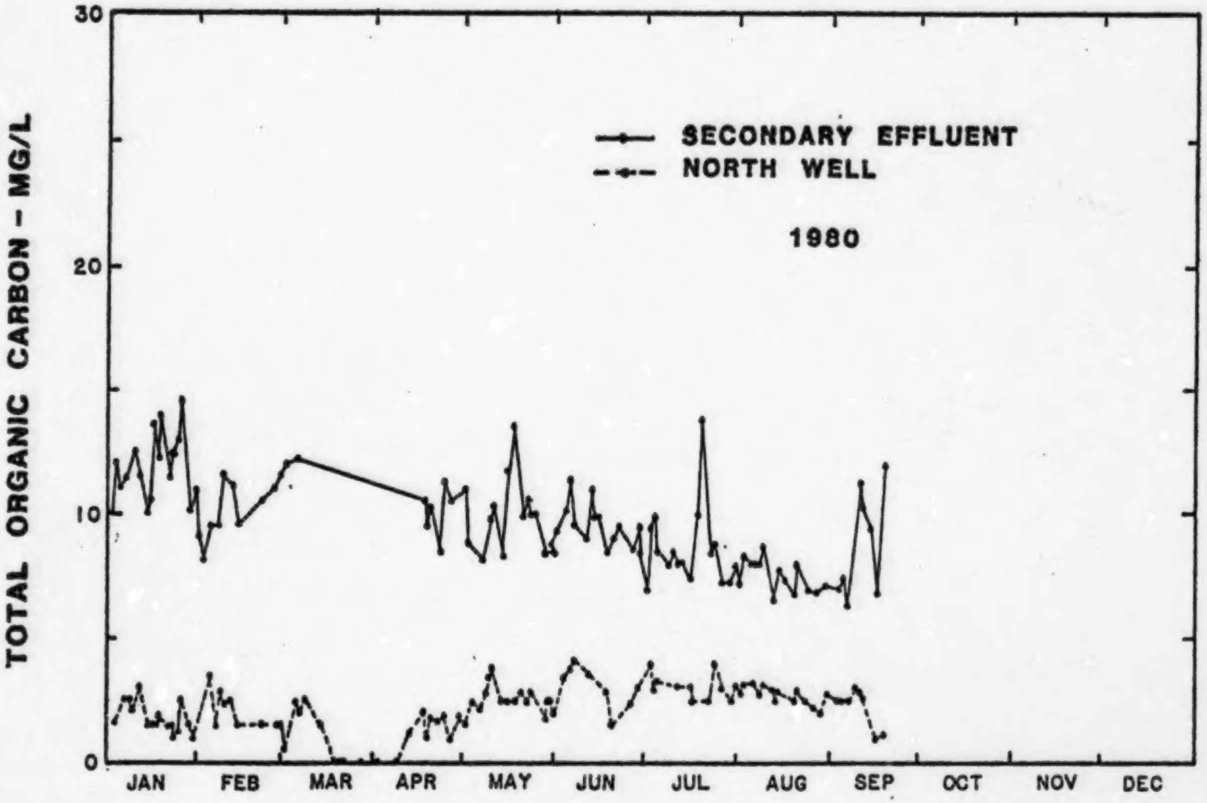

Figure 65. TOC concentrations in secondary effluent and in renovated water from the North Well, 1980.

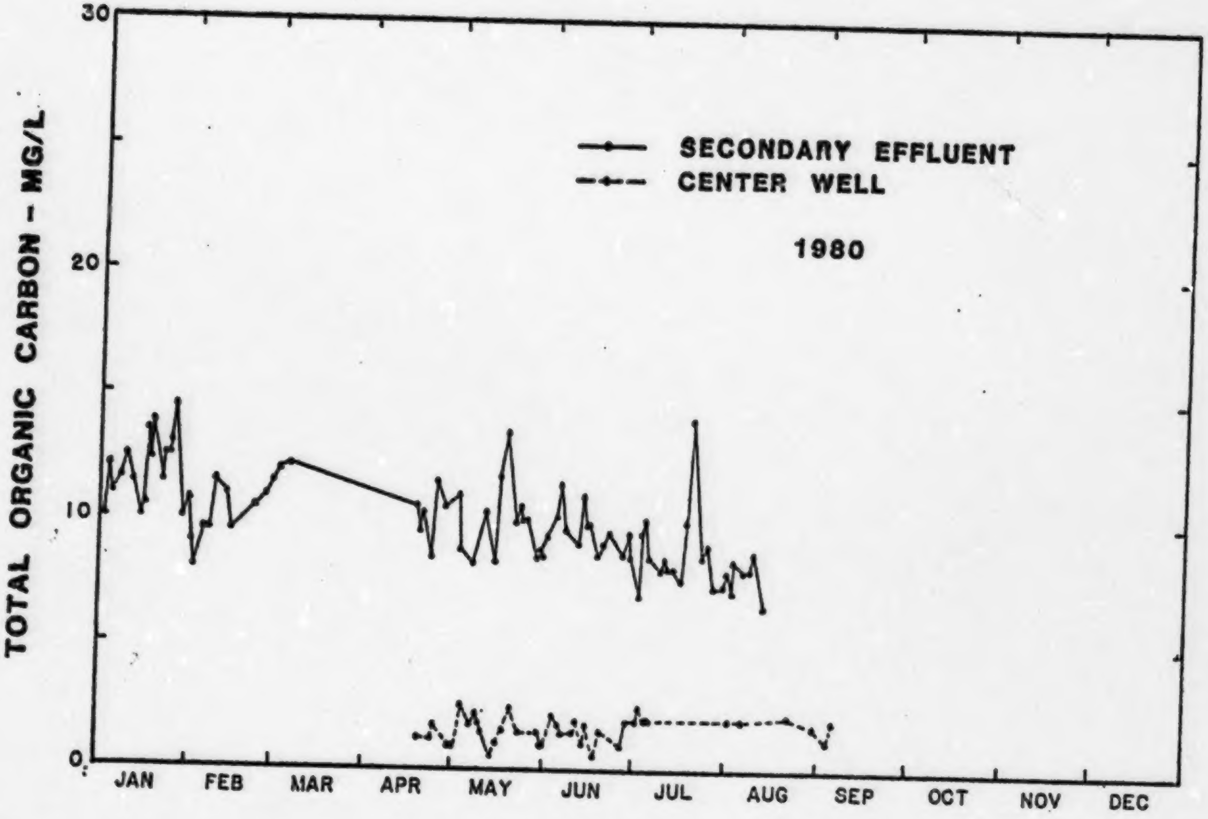

Figure 66. TOC concentrations in secondary effluent and in renovated water from the Center Well, 1980.

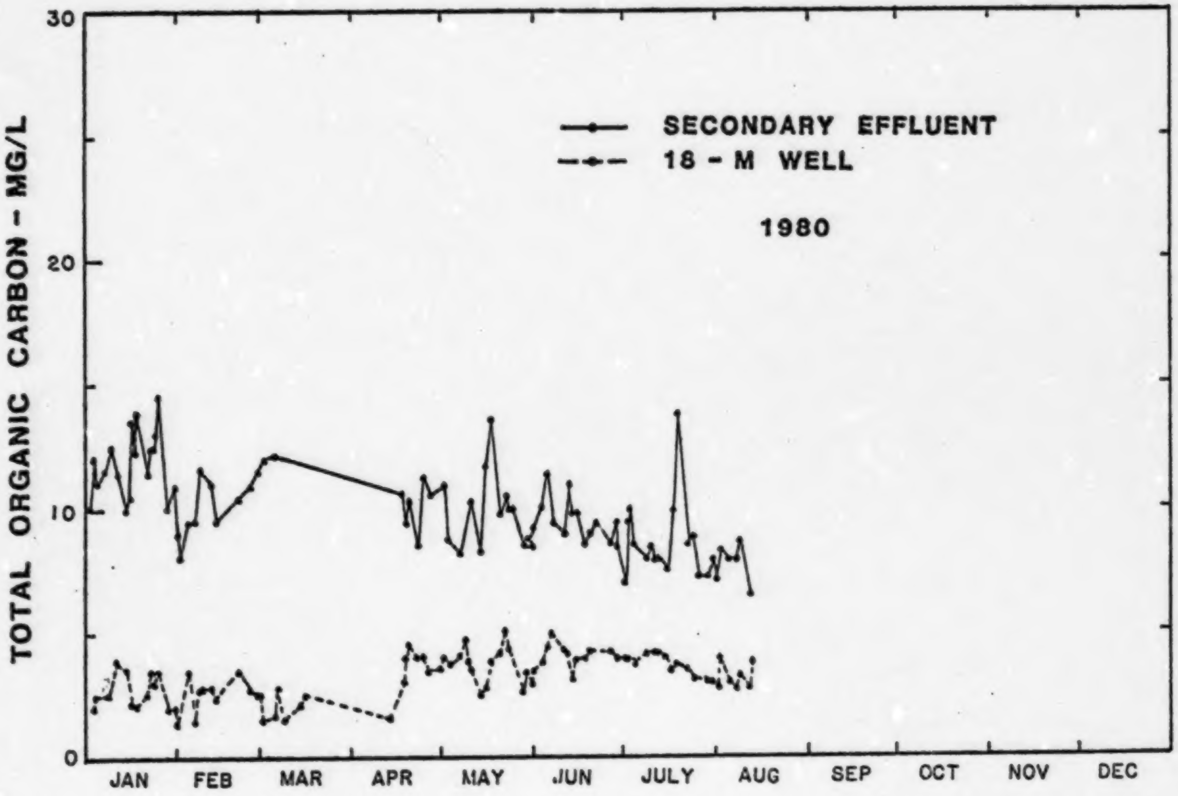

Figure 67. TOC concentrations in secondary effluent and in renovated water from the 18-m Well, 1980.

1980 by staff members of the Department of Environmental Science and Engineering of Rice University, Houston, TX (M. S. Tomson, J. Dauchy, S. Hutchins, C. Curran, C. J. Cook, and C. H. Ward) and by staff members of the Environmental Engineering and Science Program, Department of Civil Engineering of Stanford University, Stanford, CA (E. J. Bouwer, J. Graydon, and M. Reinhard).

Rice University Studies

On 24-27 January 1980, preliminary samples for trace organic analysis were obtained from the 23rd Avenue Project. The existing pump in the 18-m Well used for sampling was a 0.9 m in length, 10-cm diameter Berkeley Model #4AM8 all stainless steel (except for the hard-plastic impeller and the insulation-coated power cord) completely sealed submersible pump, rated at 38 ℓ/minute. It has been suggested that most commercial electric pumps cannot be used to obtain groundwater samples for trace level organics analysis because trace level organics from the bearings, plastic components, etc. are continuously being released to the water. To test the effects of such a pump on the sampling procedure, two samples were taken at the same time, one using the installed submersible pump and one using an all glass and Teflon nitrogen-powered pump developed at Rice University (Tomson et al., 1980). The glass and Teflon pump was lowered down the 18-m Well casing to about 1 m below the submersible pump. During the 12 hours required for sampling, the submersible pump discharged about 27 m^3 of water through a hose to basin 3. The 60-cm^3/minute flow required for the XAD-2 adsorption column was obtained via a 0.64-cm steel tee screwed into the submersible pump riser pipe about 5 to 10 cm above the casing top. Sampling consisted of passing 10 to 50 ℓ of groundwater or sewage through a 15-cm by 0.9 cm i.d. Teflon column filled with purified XAD-2 resin. A variable speed peristaltic pump (Masterflex) located after each XAD-2 resin column was used to control the water flow rate through the columns at about 60 cm^3/minute. Organic compounds were adsorbed and concentrated on Amerlite XAD-2 macroreticular resin (Rohm and Haas, Philadelphia, PA). This resin is a low-polarity, styrene-divinylbenzene copolymer with excellent surface characteristics for the adsorption of lipophilic contaminants.

A gas chromatogram of each of the extract concentrates from the columns was obtained from a Tracor 560 GC with FIDs using a 50-m fused silica Hewlett-Packard SP 2100 WCOT capillary column. The chromatograms were recorded on a Spectra-Physics 4100 integrating computer. A portion of each sample was then spiked with D_8Napththalene and mass spectral analyses were performed on a Finnigan 4000 MS using a matched GC capillary column. GC confirmation was performed by known addition or by retention time matching on as many compounds as could be obtained. In most cases, quantitation was based upon internal standards using the SP 4100 integrated peak areas. Since the results from the two pumps were quite similar, a more extensive sampling of the effluent and renovated water at the 23rd Avenue Project was made February 26 to March 1, 1980, using the commercial pumps only.

A summary of the refractory volatile organic (RVO) compounds found in the February 26-March 1 samples is presented in Table 7. The "purity" index in Column 2 is a measure of how well a library mass spectrum matches the corresponding sample spectrum. From experience with other but similar groundwater and sewage samples, it was found that the average purity index and standard deviation of compounds subsequently confirmed by GC retention time matching was about 830 ± 150. Therefore, only compounds which have been confirmed by GC retention time matching ("x" in Column 3) or have a "purity" $>$ 680, are reported. The compounds reported in Table 7 constitute 54%, by number, of compounds identified, but from 75 to 85% of the RVO mass (estimated by area under the GC trace). In addition to the RVO's listed, 3-methylpentane, 2-isopropoxy-1-propanol, ethyl acetate, chloroform, 2,4,6-trimethyl-1,3,5-trioxane, and 1,1,1-trichloroethane with purity indexes 929, 793, 916, 993, 861, and 959, respectively, were identified by mass spectral analysis to be under the solvent peak, prior to trichloroethylene.

The compounds in Table 7 are listed in the approximate order that they elute from the capillary GC column with a few exceptions of isomers which have been grouped together for illustration. The Mass Spectral Search System (MSSS) used in this study to assist in compound identification often fails to correctly differentiate between closely related isomers such as the various trialkyl-benzenes, tetralkyl-benzenes, dichloro-benzenes, methyl-napthalenes, and different high molecular weight alkanes, but for "purity" $>$ 680 rarely is the compound class or overall carbon number in error. If the compound was also confirmed by GC retention time matching, there is little uncertainty in its specific identity. An overall estimate of the uncertainty in the concentrations was made by various repetitive injections, extractions, etc. and probably is within a factor of ± 1.0 to 2.0 of the value for compounds at concentrations < 0.1 μg/ℓ to about ± 0.2 to 0.5 of the value reported for concentrations > 0.1 μg/ℓ.

For the analytical reasons discussed above and because compounds of one chemical class often have similar biological, physical, and chemical properties, the sum of compound concentrations by chemical class are listed in Table 8. The overall weighted average RVO removal was 92%. When all compounds were considered, regardless of "purity" index, the removal was from 16.578 μg/ℓ to 2.245 μg/ℓ, or 86.4%. The difference was not clearly attributable to any given class. From Table 8 it can be seen that most classes of compounds were removed from 90 to 100%. Noted exceptions were chloroalkanes, alkylphenols, alkanes, phthalates, and amides. The phthalate data may be in error because previous analyses of other sites and other data showed phthalate removals ranging from 70 to 80%. However, if the phthalate data is omitted, the Δ RVO% only increases from 92 to 94%.

Stanford University Studies

On 27-28 February 1980, samples of secondary effluent entering and leaving the infiltration basins, and of renovated water from various

TABLE 7. REFRACTORY VOLATILE ORGANICS (μg/ℓ): 23RD AVE. PROJECT, PHOENIX, ARIZONA

Compound	Purity index	GC Conf.	Sewage		Renovated water from 18-m Well				
			102	111	100	103	106	109	113
1. Trichloroethylene	947		.032	.038					
2. Toluene	958	x	.964	1.310	.017	.009	.008	.019	.024
3. Tetrachloroethylene	979	x	.251	.050	.029	.021	.034	.065	.074
4. 2,4-Dimethyl-3-hexanol	684		.011	<.008	.007	.005	.004	.006	.007
5. Chlorobenzene	764	x	.015	<.008	P*	.002	.002	.002	.002
6. 1,3,5,7-Cyclooctatetraene	933				.030	.007	.008	.015	P
7. p-Xylene	968	x	2.140	4.050	.049	.010	.040	.011	.019
8. o-Xylene	960	x	.210	.168					
9. Diethyldisulfide	979		.015	.036					
10. Propylbenzene	824	x	.056	.021					
11. m-Ethyltoluene	881	x	.234	.129					
12. o-Ethyltoluene	924	x	.105	.049					
13. 1,3,5-Trimethylbenzene	908	x	.132	.047					
14. 1,2,3-Trimethylbenzene	945	x	.128	.085					
15. 2,4,4-Trimethylbenzene	913	x	.332	.129	P				
16. m-Dichlorobenzene	947	x	.263	<.008	P				
17. p-Dichlorobenzene	943	x	.617	.355	.042	.022	.043	.072	.069
18. o-Dichlorobenzene	945	x	.986	.210					
19. 1-Methyl-4-isopropyl-7-Oxabicyclo-[2,2,1]-heptane	808		.128	.085					
20. 1-Chloro-2,3-dihydro-1H-indene	927				.025	.017	.040	.080	.070
21. 1,1'-Bicyclopentyl	910		.008	.080					
22. 2-Ethyl-1,4-dimethylbenzene	915		.008	.080					
23. 1-Methyl-2-isopropylbenzene	770	x	.008	.080					
24. 1-Methyl-4-isopropylbenzene	734	x	.086	.132					
25. Fenchone	706	x	.075	.056					
26. m- or p- Cresol	881	x	.075	.078					
27. 1,2,3,4-Tetrahydronaphthalene	773		1.490	1.980					
28. Camphor	582	x	.164	<.008					

(continued)

TABLE 7 (continued)

Compound	Purity index	GC Conf.	Sewage 102	Sewage 111	Renovated water from 18-m Well 100	103	106	109	113
29. 1,6-Dichloro-1,5-cyclooctadiene	791				.011				
30. 2,3-Dihydro-1H-inden-1-one	866		.087	.099					
31. 1-(4-Ethylphenyl)ethanone	716		.006	.026					
32. p-Isopropylphenol	491	x	.061	.028					
33. α-Terpineol	581	?	.045	<.008					
34. 2-Methylnaphthalene	581	x	.226	.396					
35. 1-Methylnaphthalene	811	x	.050	.059					
36. 3,3-Dimethylhexane	850		IP**	IP	.012	.004	.009	.011	.009
37. p-t-Butylphenol	748		.125	IP					
38. p-t-Butylphenol	722		.150		.118	.040	.048	.069	.046
39. m-t-Butylphenol	849	x	.189	.116	P				
40. p-t-Butylphenol	738		.819	.907	.069	.035	.090	.097	.074
41. 1H-Indole	662	x	.009	.086					
42. Skatole	901		.030	.031					
43. Undecane	773					.004			
44. 3,5,24-Trimethyltetracontane	742		.095	.110					
45. Dimethylphthalate	585	x	.023	.015					
46. 1,2-Dihydro-2,2,4-tri-methylquinoline	720				.093	P	.019	.021	.056
47. 2,6-Di-t-butyl-p-benzoquinone	760		.022	.037	.005	P	P	P	P
48. O-Decylhydroxxylamine	687		.005	<.008					
49. 2,6-Di-t-butyl-p-cresol	887		.267	.250	.056	.200	.071	.096	.079
50. t-Butyl-4-methoxyphenol	687		.008	.131					
51. α-Hydroxybenzeneacetic acid	779		.008	.082					
52. 1,6-Dioxacyclododecane-7,12-dione	735				.021		P	P	P
53. o-Phenylphenol	751		.008	<.008					
54. Butoxymethylbenzene	760				P	.005	.007	.008	.005
55. Nonylbenzene	808		.008	.680?					
56. Diethylphthalate	905	x	.231	<.008	.017	P	P	P	P
57. 2-Methylthiobenzothiazole	711	x	.193	.202	.028	.004	.023	.016	.011
58. Benzophenone	894	x	.233	.285	.045	.016	.014	.014	.009
59. 2-Propyl-1-heptanol	803				.005	.003	.012	.008	.006
60. p-(1,1,3,3-Tetramethylbutyl)phenol	923	x	.757	.809	.017	.010	.014	.013	.012

(continued)

TABLE 7 (continued)

Compound	Purity index	GC Conf.	Sewage 102	Sewage 111	Renovated water from 18-m Well 100	103	106	109	113
61. p-(2,2,3,3-Tetramethylbutyl)phenol	896		.156		.103	.107	.053	.072	.042
62. N-(3-Methylphenyl)acetamide	773		IP	IP	.110	.114	.030	.041	.025
63. Hexadecane	703		.052	.057					
64. Hexatriacontane	852		.035	.036	.130	.271	.095	.028	.090
65. Hexatriacontane	850		.120	.022	P	P	P	P	P
66. 4,6,8-Trimethyl-1-nonene	849					.013			
67. Dibutylphthalate	957		.248	.260	.316	.729	.274	.314	.239

* P - indicates compound was present, but at too low concentration to estimate.

** IP - indicates that the corresponding GC and GC/MS peak could not be reliably separated from background peaks.

TABLE 8. REFRACTORY VOLATILE ORGANICS (μg/ℓ) BY CLASS, 23RD AVE. PROJECT, PHOENIX, ARIZONA

Class (Typical Example)	Purity > 680 Average sewage	 Average 18-m W	ΔRVO%
1. Chloroalkanes (tetrachloroethylene)	0.136	0.056	70
2. Chloroaromatics (p-dichlorobenzene)	0.922	0.055	94
3a. Alkylbenzenes (o-xylene)	5.943	0.123	98
3b. Alkylphenols (p-isopropylphenol)	1.624	0.241	85
4. Alkylnaphthalenes (2-methylnapthalene)	2.101	0.000	100
5a. Alkanes (hexatriacontane)	0.671	0.193	71
5b. Alkohols (2,4-dimethyl-3-hexanol)	0.132	0.006	95
5c. Ketones (2,6-d-t-butyl-p-benzoquinone)	0.197	0.004	98
6. Phthalates (dibutylphthalate)	0.385	0.379	2
7. Indoles, Indenes (IH-indole)	1.736	0.071	96
8. Amides (N-[3-methylphenyl]acetamide)	0.212	0.055	74
9. Alkoxyaromatics (butoxymethylbenzene)	0.107	0.010	91
TOTAL	14.216	1.193	

ΔRVO% $\frac{\sum \text{Sew} - \sum \text{18-m W}}{\sum \text{Sew}} \times 100\%$

wells were taken and shipped to Stanford University for volatile organic analysis (VOA: see Henderson, et al., 1976), closed-loop stripping analysis (CLSA: see Grob and Zurcher, 1975), and hexane solvent extraction analysis (SEA: see Reinhard, et al., 1979). Results of the low molecular weight halogenated organic analyses (volatile organic analysis or VOA) are shown in Table 9. Compounds at or below the detection limit of 0.1 μg/ℓ were:

Carbon tetrachloride	CCl_4
Bromodichloromethane	$CHBrCl_2$
Dibromochloromethane	$CHBr_2Cl$
Bromoform	$CHBr_3$

Traces of these compounds were detected, but they were insignificant and/or due to possible solvent contamination.

The VOA results (Table 9) indicate that volatilization losses are probably the major removal mechanism for these compounds. Removals of at least 60% were observed between infiltration basin inflow (secondary) effluent) and basin outflow. The infiltration basin outflows had lower concentration fluctuations than the inflows. Thus, the basins appeared to damp out some of the influent concentration variations. The halogenated organic concentrations in the 18-m Well and 30-m Well were similar to the basin inflow concentrations. Thus, there was little or no reduction in concentration of these halogenated organics during soil passage. Concentrations in the water from the North Well, however, were considerably lower than in that from the 18-m and 30-m Wells, either as a result of sorption, degradation, or dilution by Salt River water (flooding period).

Chloroform showed the greatest reduction among the 4 compounds. For example, chloroform in North Well water was lower than in basin outflows, whereas, the other 3 compounds were higher in North Well water. Variations in basins 1 and 4 inflow concentrations probably account for the high values of 1,1,1-trichlorethane and tetrachlorethylene found in the 18-m Well and the 30-m well.

The compounds identified by CLSA were primarily hydrocarbons, alkylated benzenes, and a few chlorinated benzenes (Table 10). Concentrations in the water leaving the basins were significantly lower than in water entering the basins. This was probably due to volatilization losses from the infiltration basins, as for the VOA compounds. Most of the CLSA compounds identified in basin effluents also appeared in the observation wells, but at lower concentrations. The longer underground travel to the North Well did not significantly alter the concentrations compared to those for the 18- and 30-m Wells in the center. Many of the alkylated aromatics are biodegradable, and are suspected to be removed in the system by biological action as evidenced by the low concentrations in the water from the 18-m and 30-m Wells. The mass spectra of several C_3-substituted benzenes (C_9H_{12}) were not differentiated, so no further identification could be achieved.

TABLE 9. RESULTS OF VOLATILE ORGANIC ANALYSIS (VOA), µg/ℓ

Sample	Chloroform $CHCl_3$		1,1,1 Trichloroethane Cl_3CCH_3		Trichloroethylene $Cl_2C=CClH$		Tetrachloroethylene $Cl_2C=CCl_2$	
	Samples	Ave.	Samples	Ave.	Samples	Ave.	Samples	Ave.
Basin 1 inflow 2/28/80	4.92 5.19	5.06	4.08 4.48	4.28	1.54 1.73	1.64	6.44 7.40	6.92
Basin 1 outflow 2/28/80	1.28 1.30	1.29	1.98 1.81	1.90	0.23 0.36	0.30	2.09 1.78	1.94
Basin 4 inflow 2/29/80	4.47 5.33	4.90	7.50 10.1	8.80	1.33 1.76	1.55	7.27 11.3	9.29
Basin 4 outflow 2/29/80	1.60 1.69	1.65	1.78 1.76	1.77	0.33 0.34	0.34	1.31 1.15	1.23
North Well 2/28/80	0.32 0.32	0.32	1.92 2.22	2.07	0.89 1.05	0.97	3.20 3.46	3.33
North Well 2/29/80	0.25 0.27	0.26	3.16 3.06	3.11	0.87 0.88	0.88	2.98 2.65	2.82
18-m Well 2/28/80	1.85 1.86	1.86	8.55 8.30	8.43	1.73 1.71	1.72	10.8 10.6	10.7
18-m Well 2/29/80	1.73 1.74	1.74	10.1 8.24	9.17	1.96 1.64	1.80	13.7 10.3	12.0
30-m Well 2/28/80	2.13 1.99	2.06	6.29 5.98	6.14	2.16 2.52	2.34	10.8 11.7	11.3

TABLE 10. RESULTS OF CLOSED-LOOP STRIPPING ANALYSIS (CLSA). ng/ℓ

Compound	Basin 1 2/28/80		Basin 4 2/29/80		18-m Well 2/28/80	18-m Well 2/29/80	30-m Well 2/28/80	North Well 2/28/80	North Well 2/29/80
	Inflow	Outflow	Inflow	Outflow					
+Ethylbenzene C_8H_{10}	1890	165	2285	355	20	60	85	35	205
meta and para xylene C_8H_{10}	13,700	385	3610	440	45	140	245	110	995
Unknown	500	70	1535	250	—	—	—	—	—
ortho xylene C_8H_{10}	5400	185	2685	—	80	120	215	120	515
normal dodecane $C_{12}H_{26}$	1580	140	1970	400	—	—	—	—	—
Unknown (Iso-butylmethyl acetal $C_7H_{16}O_2$?)	—	—	—	—	115	215	190	15	60
2,2,4-trimethyl heptane $C_{10}H_{22}$	1330	335	2685	555	—	—	15	—	—
C_3-benzene C_9H_{12}	2890	155	2565	—	20	100	155	125	545
C_3-benzene C_9H_{12}	1985	95	2375	90	30	40	70	25	130
Unknown	1305	65	1500	—	30	50	55	20	100
C_3-benzene C_9H_{12}	4260	145	2380	—	25	85	100	40	455
ClC_8H_{17} (internal std.)	880	480	1060	555	240	235	175	190	175
C_3-benzene C_9H_{12}	1810	90	1400	—	35	45	60	20	155
+1,3-dichlorobenzene $C_6H_4Cl_2$	2020	340	1410	285	290	305	430	210	310
+1,4-dichlorobenzene $C_6H_4Cl_2$	3170	635	3255	945	890	980	1265	650	795
+1,2-dichlorobenzene $C_6H_4Cl_2$	4480	685	3415	710	1300	1440	1810	1015	1195
Unknown	155	—	505	40	20	40	45	30	85
+1,2,4-trichlorobenzene $C_6H_3Cl_3$	1150	—	1100	300	315	225	270	180	215
+naphthalene $C_{10}H_8$	940	150	785	260	395	210	590	170	410
$ClC_{12}H_{25}$ (internal std.)	500	500	500	500	300	300	200	200	200
2,2,4-trimethyl-penta-1,3-diol-isobutyrate $C_{10}H_{30}O_4$	645	120	570	270	165	480	160	75	70
Unknown	—	—	—	—	105	130	105	—	—
Unknown	835	105	285	250	160	360	330	100	145
Unknown	—	—	—	—	635	610	115	—	—
$ClC_{16}H_{33}$ (internal std.)	220	375	115	225	200	810	140	105	120

+ Quantified using response factors determined with standards.
All other concentrations are relative to the internal standard $ClC_{12}H_{25}$ (response factor = 1.0).

— Below detection limit of 10 ng/ℓ.

The chlorinated benzenes and naphthalene showed little reduction during soil percolation. Although these compounds have been found biodegraded in aerobic column studies at Stanford University, no anaerobic conditions were found under which these compounds were degraded (unpublished results). With the high organic loading during infiltration, it is likely that most of the soil profile was anaerobic except for a small aerobic zone at the soil surface. Therefore, significant biological removal of the chlorinated benzenes and naphthalene in the anaerobic aquifer would not be expected. This also accounts for further removals with longer underground travel to the North Well.

Compounds identified by solvent extraction analysis (SEA) included (Table 11):

phthalates	- plasticizers
chlorinated anisoles	- possible methylation products of chlorophenols
DDT, lindane	- pesticides

These compounds are not as volatile or biodegradable as the compounds measured by VOA and CSLA, and many of them appeared in the observation wells. Significant reduction in concentration between basin inflows and outflows occurred for some of the compounds identified by SEA. Some were also reduced during passage through the soil. This was most likely due to sorptive processes. However, the compounds generally were present at concentrations near the detection limit, except for the phthalates, making it difficult to make positive conclusions about observed removals.

A more systematic study of the trace organics in the sewage effluent and renovated water was begun in September 1980, sampling on two days each week. For the effluent, samples were obtained at different times of the day and at different points in the infiltration basins. The trace organics identification again was done by Stanford University, in cooperation with the Western Regional Research Center laboratory of the U. S. Department of Agriculture (SEA-AR) in Albany, California. The first series of samples and analysis were done to characterize the trace organics before chlorination of the effluent, which started routinely at the treatment plant in the middle of November. Starting in April 1981, a second series of 2-month sampling will be done to see if there is any effect of the chlorination on the trace organics in the plant effluent and in the renovated water from below the infiltration basins. The results of these studies will be presented in one report next year.

TABLE 11. RESULTS OF SOLVENT EXTRACT ANALYSIS (SEA). ng/ℓ

Compound	Basin 1 2/28/80		Basin 4 2/29/80		18-m Well	18-m Well	30-m Well	North Well
	Inflow	Outflow	Inflow	Outflow	2/28/80	2/29/80	2/28/80	2/28/80
Unknown	1420	65	2170	330	25	50	25	—
Unknown	120	170	490	160	20	10	10	—
+2,3,5,6-tetrachloroanisole $C_7H_4OCl_4$	—	—	—	—	35	—	—	—
+diethyl phthalate $C_{12}H_{14}O_4$	8830	12,700	35,300	8620	8610	4550	9500	1560
+2,3,4,5-tetrachloroanisole $C_7H_4OCl_4$	—	—	—	—	20	—	20	—
+pentachloroanisole $C_7H_3OCl_5$	—	—	—	—	730	70	205	35
+lindane $C_6H_6Cl_6$	20	30	75	20	15	—	—	—
Unknown	1080	10	1850	55	—	—	—	—
diisobutyl phthalate $C_{16}H_{22}O_4$	45	20	80	—	30	—	—	—
Unknown	1880	135	3000	630	45	35	50	—
Unknown	85	15	115	50	130	55	100	35
Unknown	185	115	280	100	30	—	15	35
Unknown	25	—	—	20	35	35	35	50
+DDT $C_{14}H_9Cl_5$	10	10	20	—	10	—	10	—
hexabromobenzene (Internal Std) C_6Br_6	200	200	200	200	100	200	100	100
Unknown	—	185	150	45	35	80	80	—
+diethylhexyl phthalate $C_{24}H_{38}O_4$	1030	1140	5230	950	625	425	790	570

+ Quantified using response factors determined with standards.
All other concentrations are relative to the internal standard hexabromobenzene.

— Below detection limit of 10 ng/ℓ.

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